AN INVESTIGATION TO DETERMINE THE ECONOMY AND PRACTICALITY OF USING VARIOUS TYPE SOILS TREATED WITH PORTLAND CEMENT OR OTHER ADMIXTURES FOR HIGHWAY CONSTRUCTION

by

Radnor J. Paquette
[and others]

Project No. B-136 [HPS-1(54)]

Contract with the State Highway
Department of Georgia
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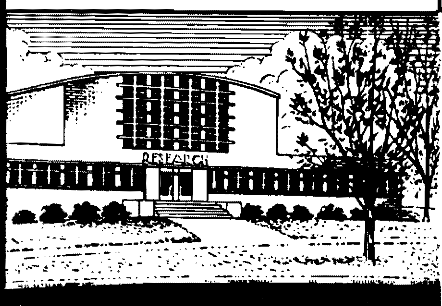

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PREFACE

Test results of Soils I, II and III combined with portland cement were previously reported as Soils C, D and E in the Annual Report No. 1, Project B-136 [HPS-1(54)] submitted in February, 1959.

Appreciation is expressed to the members of the State Highway Board of Georgia for their interest in this work. Special credit is due to Mr. M. L. Shadburn, State Highway Engineer, for promoting research; to Mr. Roy A. Flynt, State Highway Planning Engineer, for his aid in arranging the many details; and to Mr. W. F. Abercrombie, State Highway Materials Engineer, for his assistance in planning the method of attack.

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ABSTRACT

In order to meet the demands of an expanding economy and a tremendous increase in vehicles, highway engineers are confronted with the problem of building and maintaining better highways which will withstand the greater traffic and increased loads. These high quality highways require greater stability and strength in the load supporting portions of the roadway. In many locations, suitable soil for the base course and subgrade is not available, creating a problem of transporting suitable soils to the area or changing the physical properties of the available soils by stabilization with admixtures.

This work was undertaken to study various soils and evaluate the effectiveness of stabilization with various admixtures. Used in this study were five selected soils of widely varying physical properties. Using compressive strength to evaluate stability, these soils were stabilized with portland cement, a lime and flyash mixture, phosphoric acid, and asphaltic cutback, RC-3.

Each soil was mixed with the individual admixtures and moisture-density tests were performed to determine the effect on the maximum dry density and optimum moisture. Using this density and moisture data, samples 2.8 inches in diameter by 5.6 inches in height were statically compacted and cured for 7 and 28 days. These samples were then tested by an unconfined compression test and a triaxial test using a lateral pressure of 20 psi. Additional samples of the soil-portland cement mixture were tested with a lateral pressure of 50 psi for plotting Mohr's

diagrams to determine the effect of that stabilizer on the "angle of internal friction" and "cohesion."

Results of the study indicate that phosphoric acid slightly increased the density in all soils. Portland cement, lime-flyash and RC-3 increased the density in the uniformly graded soils. There was little effect from the addition of portland cement or RC-3 in the well-graded soils while lime-flyash caused a marked reduction in density in these soils.

Strength tests indicated that portland cement was the most effective stabilizer in all soils giving high strength gains. The addition of portland cement also increased the "angle of internal friction" and "cohesion." The lime-flyash admixture and phosphoric acid caused slight strength increases in all soils. Some soils had a negligible strength increase with the addition of RC-3, while other soils indicated a reduction in strength.

CHAPTER I

INTRODUCTION

General.--Soil stabilization with portland cement and other admixtures has become of great importance in recent years. In order to stay abreast of the expanding economy and tremendous increase in vehicles and vehicle-miles traveled each year, highway engineers are confronted with the problem of building and maintaining more and better roads. This problem is not only concerned with shorter and faster routes but also with roads which must be able to withstand the loads imposed by larger and heavier truck traffic. Stabilization has aided the engineer in solving these problems.

A basic requirement in constructing high quality roads is providing a base course of sufficient strength to distribute the high intensity load applied to the pavement to a smaller stress which can be supported by the weaker subgrade. For the stability necessary for this load distribution, a base course soil mixture should be composed of aggregate which is strong and durable enough to resist weathering and crushing, and soil fines of a character such as to provide graded mixtures with sufficient cohesion to act as a binder but without the risk of detrimental volume change. Some areas do not have an available supply of soil meeting the above requirements; therefore the engineer is confronted with the problem of transporting suitable soil into this area or attempting to change the characteristics of the available soil by artificial methods. This artificial changing of the physical properties of a soil is termed "soil stabilization."

This research was undertaken to study various soils and to evaluate the effectiveness of stabilization with various admixtures. The five soils selected were typical soils found in Georgia. The admixtures chosen for comparative purposes were portland cement, asphaltic cutback, phosphoric acid, and a combination of lime and flyash. The use of portland cement as a stabilizing agent has increased tremendously since the first controlled soil-cement project was constructed in South Carolina about 1933, and it is probably the most widely used admixture today. Bituminous materials have had considerable use as stabilizers, especially in fine sands. The cutback, RC-3, used in this study has been successfully used in many areas. Phosphoric acid is a relatively new product in the field of stabilization but it has shown some stabilizing qualities in experimental work. The combination of lime and flyash has shown some success in this field but it, too, is relatively new.

The criteria used to evaluate these admixtures was the compressive strength, which was determined by both an unconfined compressive test and a triaxial test using a lateral confinement of 20 psi. The tests were performed on the samples after a curing period of 7 and 28 days. Additional work was done with the various soils combined with portland cement to evaluate effects of this admixture on cohesion and angle of internal friction.

Previous studies.--In 1935, the Portland Cement Association began a program of investigation in an attempt to determine the basic principles controlling mixtures of soil and portland cement. This basic research by Catton (1)* on various soils mixed with cement to produce satisfactory

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*Numbers in parentheses refer to the corresponding numbers in the bibliography.

durability and stability was based on wet-dry and freeze-thaw tests. Conclusions from that work indicated certain soil characteristics were necessary, namely:

1. Liquid Limit must be below 50 per cent.
2. Plastic Index must be below 25 per cent.
3. Clay content must not exceed 35 per cent.
4. Percentage of solids at maximum density must be 60 or greater.
5. The particular soil must possess a "regular" moisture-density curve.

Later, work by Winterkorn (2) showed that theoretical and experimental evidence permits the conclusion that satisfactory waterproofing and cementing can be accomplished with soils not previously recommended for soil-cement practice.

Felt (3) described the factors that have a pronounced influence on the physical properties of soil-cement as the soil type, quantity of water and cement added, density to which the mixture is compacted, mixing time and degree of pulverization of the soil. Goecker, et al. (4) described a study of several variables on the unconfined compressive strength of lime-flyash stabilized soils, the effect of the mixture on standard Proctor moisture-density relationship and an evaluation of the resistance of lime-flyash stabilized soils to freezing-thawing and wetting-drying. Minnick and Williams (5) described several field projects of lime-flyash soil mixtures with a comparison of performance and properties of the mixtures.

The American Road Builders' Association Committee on Soil-Asphalt Stabilization (6) discussed the uses of various asphaltic products in stabilizing sandy and cohesive soils. They described the different problems involved in stabilizing the two types of soils and suggested specifications and construction procedures. Benson (7) reported on the proper use of bituminous materials for soil stabilization. Lyons (8) described some work with phosphoric acid as a stabilizer. This work showed that soil stabilized with about two per cent phosphoric acid became less plastic, was easier to mix and increased the strength of the mix.

With this and other research work as a background, this study was undertaken to evaluate the comparative stabilizing qualities of these four admixtures with several different soils.

CHAPTER II

MATERIALS AND TEST EQUIPMENT

Soils.--The soils chosen for this study are typical of the available roadbuilding soils in the general area from which they were obtained. All soils were obtained from within the state of Georgia. Soil I is a brownish, well-graded, clayey, silty sand; Soil II is a reddish brown, uniform, silty, clayey sand; Soil III is a greyish-white uniform sand; Soil IV is a red, well-graded, silty, sandy clay; and Soil V is a yellowish-brown, well-graded clayey, silty sand. A description of the soils is given in Table 1, with the grain size distribution shown in Figure 1.

According to the Georgia Highway Department classification and usage, only Soil II would be suitable for base construction without treatment with aggregate or an admixture. Soils I, III and IV would be suitable for subgrade construction without treatment while Soil V would require treatment before using as a subgrade and would not normally be used for base construction even with treatment. It is noted from Table 1 that although Soils I and II have the same classification under the Bureau of Public Roads Classification, these two soils are vastly different in appearance, texture and stabilizing characteristics. Physical testing of Soil V indicates a granular material, but this soil is a disintegrated rock soil which is very soft and the granular structure is easily broken down which makes this a very poor soil for road construction.

Table 1. Description of Soils

Soil No.	I	II	III	IV	V
Location by County	Carroll	Effingham	Camden	Fulton	Fulton
Textural analysis % retained by wt.					
Sieve No. 10	3	0	0	3	2
Sieve No. 40	14	54	2	19	24
Sieve No. 60	37	68	7	28	36
Sieve No. 100	44	74	53	37	46
Sieve No. 200	62	83	92	46	55
Total Silt, %	21	2	3	22	24
Total Clay, %	6	11	--	27	14
Specific Gravity	2.67	2.63	2.69	2.70	2.69
Liquid Limit	13	14	--	29	37
Plastic Limit	--	--	--	23	--
Plastic Index	NP	NP	NP	6	NP
BPR Classification	A-2-4(0)	A-2-4(0)	A-3-(0)	A-4-(4)	A-4-(2)
Ga. Hwy. Dept. Classification	C-1 Topsoil	A-1 Topsoil	A-1 Subgrade	1-B Embankment	II-A Embankment

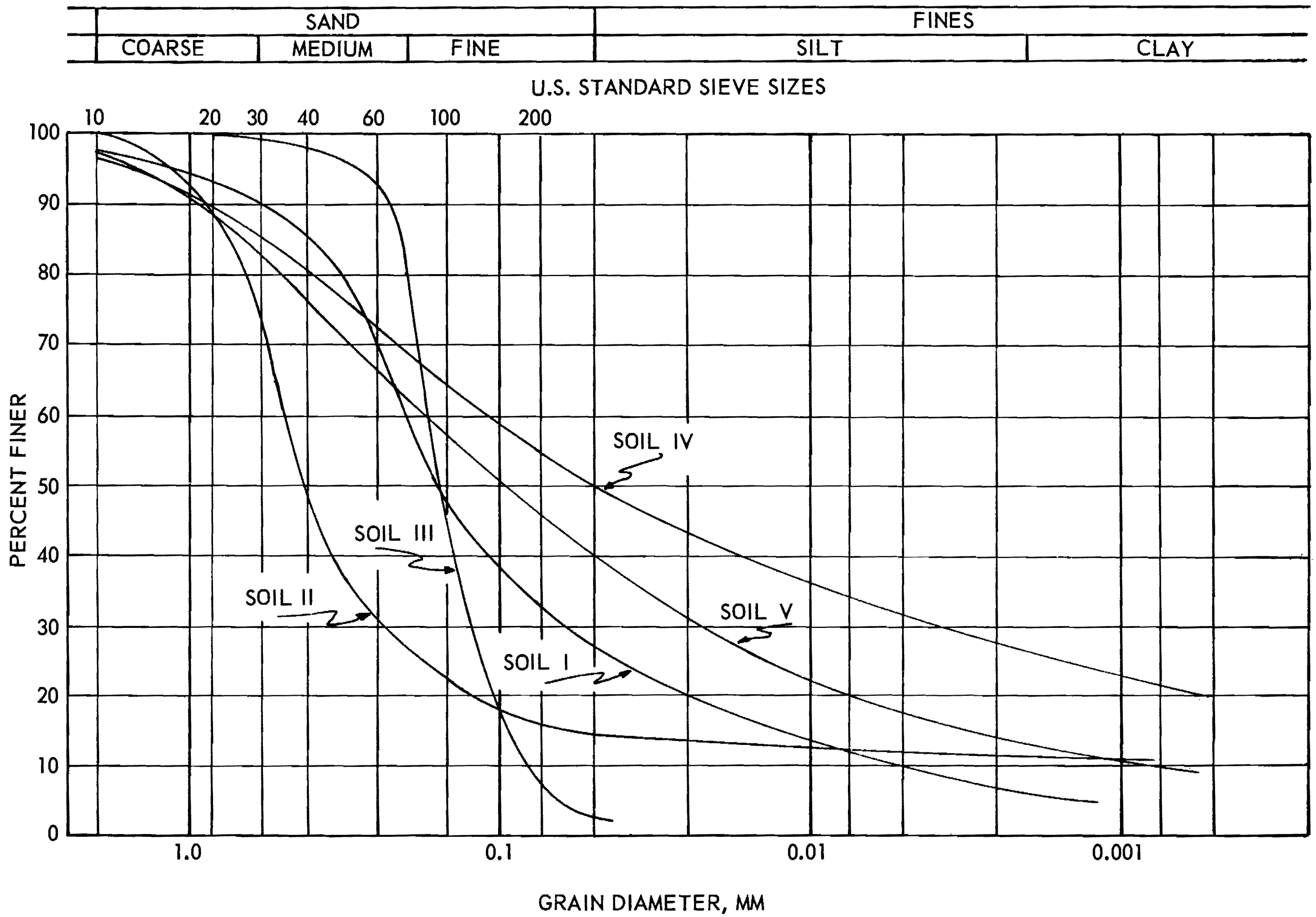


Figure 1. Grain Size Distribution.

Admixtures.--The portland cement used was Type I normal purchased on the open market. A typical analysis of the several sacks used is shown in Table 2.

The asphalt used in the study was an asphaltic cutback RC-3.

The lime used in the mixture of lime and flyash was a hydrated high calcium lime purchased on the open market. Analysis of the flyash is shown in Table 3.

The phosphoric acid used was an 85 per cent solution.

Test Equipment.--The moisture-density tests were performed with the Standard Proctor compaction equipment consisting of a mold of 1/30 cubic foot volume and compacted with a 5.5 pound hammer falling 12 inches with the soil compacted in 3 layers with 25 blows on each layer.

The molding equipment consisted of an eight inch length of steel tubing bored to 2.8 inches in diameter with a 3 inch extension attached to the top to retain the loose mixture and a split 3 inch spacer on the bottom to support the mold while filling. Compacting the mixture in the mold was a 4 inch removable piston used in the bottom and a 7 inch piston for the top which was attached to the upper movable head of the testing machine. A dial gage was mounted on the end of a measured rod for determining the proper height of the compacting mix. Application of load for compaction was from a 120,000 pound constant-strain testing machine. The molding equipment and molding processes are shown in Figures 2 and 3 respectively.

For strength determination, the sample was placed in a standard type triaxial cell. Lateral pressure, when used, was provided by compressed air metered into the sealed cell. Load was applied through a

Table 2. Portland Cement Analysis

Chemical Composition, %	
Silicon dioxide, SiO_2	20.46
Ferric oxide, Fe_2O_3	2.44
Aluminum oxide, Al_2O_3	5.90
Sulphur trioxide, SO_3	2.08
Calcium oxide, CaO	62.87
Magnesium oxide, MgO	4.18
Insoluble residue	0.30
Loss on ignition	1.38
Specific surface area, Blaine (sq. cm/gm)	3464

Table 3. Flyash Analysis

	Macon, Ga.	Columbia, S. C.
Chemical composition, %		
Silicon dioxide, SiO_2	41.40	45.92
Aluminum oxide, Al_2O_3	21.05	32.00
Ferric oxide, Fe_2O_3	8.65	16.50
Magnesium oxide, MgO	5.36	1.40
Sulphur trioxide, SO_3	1.16	0.84
Carbon, C	1.66	2.32
Loss on ignition	3.12	2.24
Specific surface area, Blaine (sq. cm/gm)	3427	1760

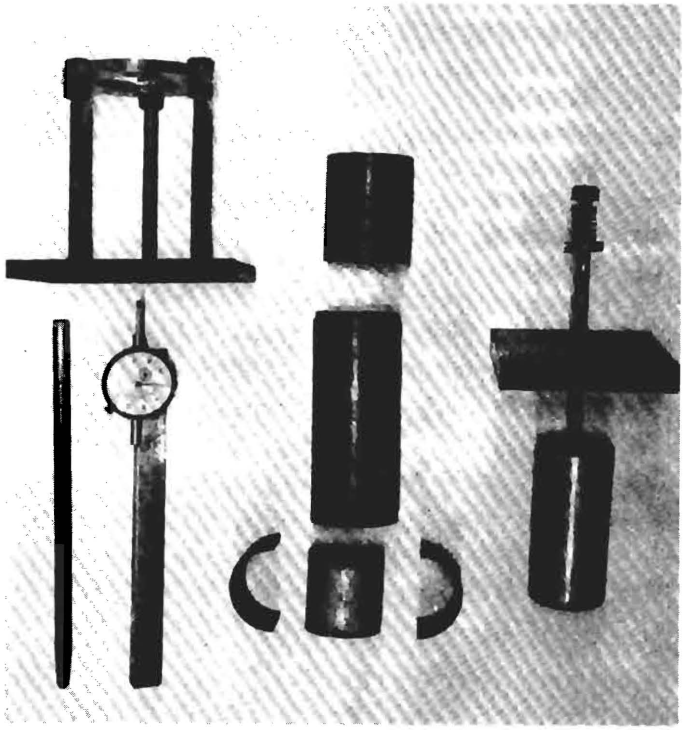


Figure 2. Molding Equipment.

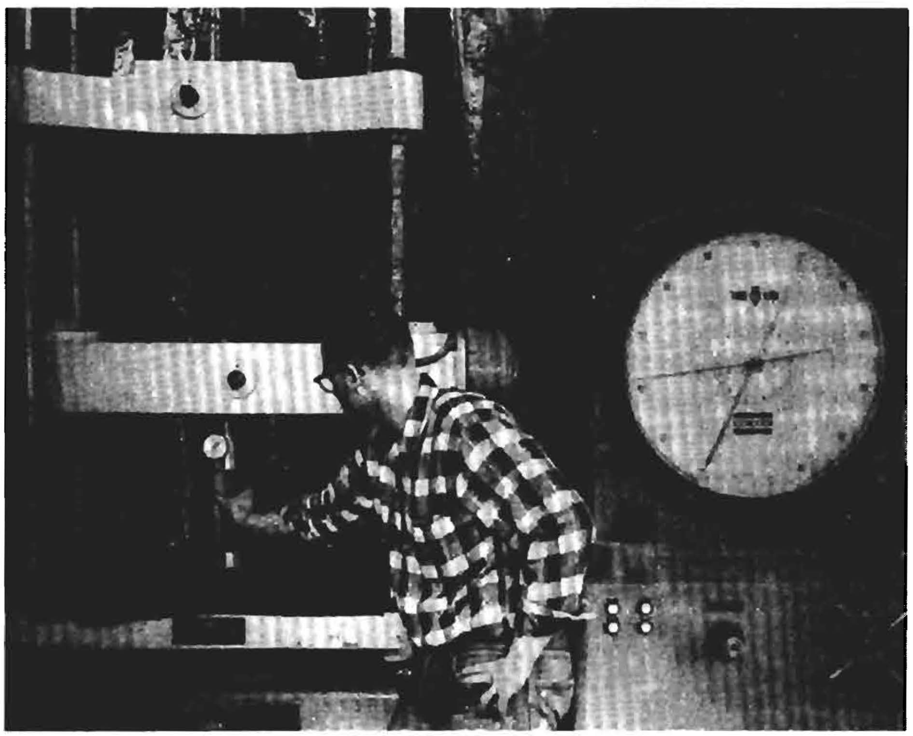


Figure 3. Molding Soil-Specimen.

piston in the top of the cell by a 30,000 pound constant strain type testing machine, or in some cases, samples were tested in the standard triaxial cell using a constant stress scales-type loading device. The constant strain and constant stress triaxial testing equipment is shown in Figures 4 and 5 respectively. Strain measurements were made with a dial gage attached to the top of the triaxial cell.

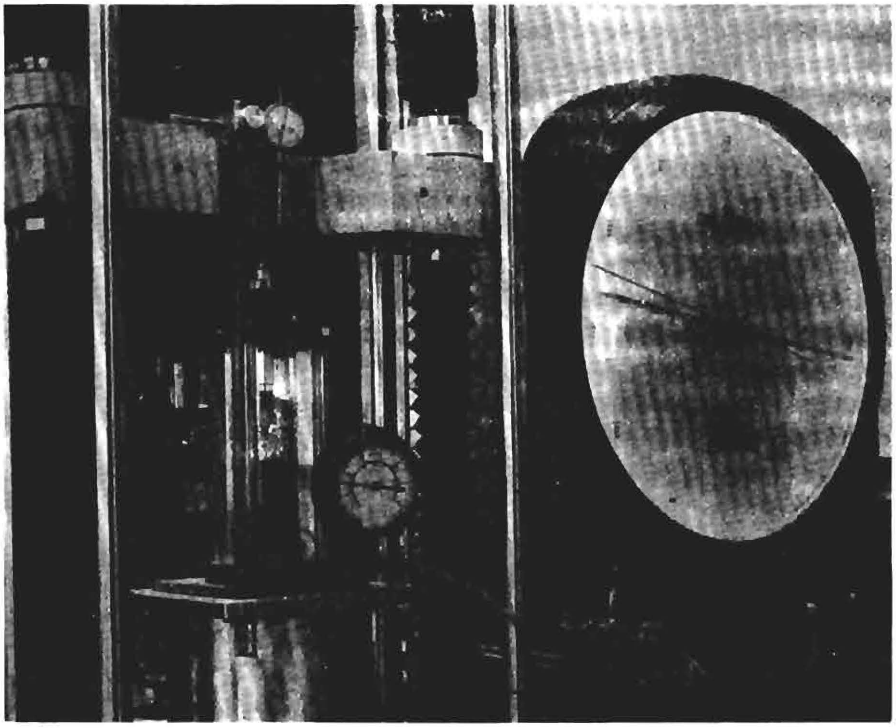


Figure 4. Constant-Strain Triaxial Equipment.

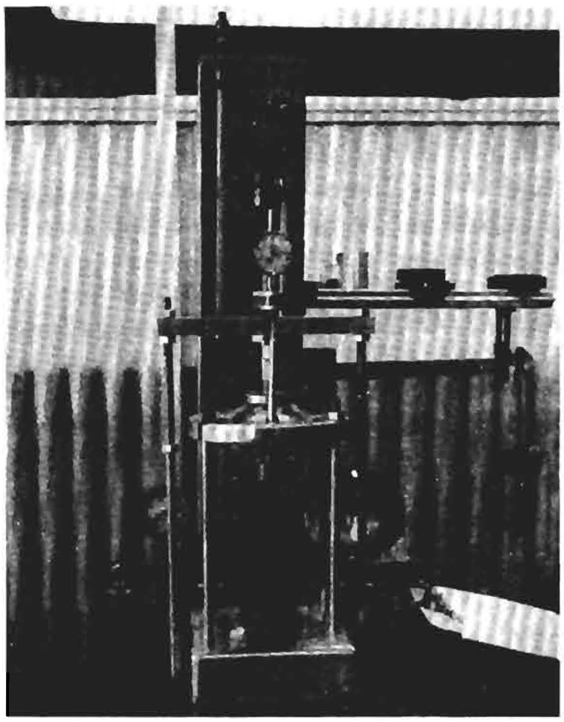


Figure 5. Constant-Stress Triaxial Equipment.

CHAPTER III

TESTING PROCEDURES

General.--The basic testing program was designed to evaluate the compressive strength of different types of soils mixed with various admixtures. The procedures adopted were for testing material passing a No. 4 sieve. Some of the desirable features in a testing program of this nature include:

1. A standard size sample and method of compaction which is suitable for testing various type soils.
2. A method of curing which is comparable to field curing conditions.
3. Evaluation by testing the stability of the samples under conditions which can be correlated to actual field conditions of stability failures.
4. Consistency in reproducing test results.

Preparation of soil and mixing.--The soil was air-dried to a uniform moisture content and sieved through a No. 4 sieve with only the material passing being used in the tests. All the soils were predominantly minus 4 materials, with the majority of the discarded material being hardened lumps and roots.

Mixing was done in a Hobart Model C-100 mixer at a speed of 144 RPM. For soils with a dry mixture, the dry ingredients were mixed one minute, the water for the proper moisture content was added and then mixed for 9 minutes. For the mixture with phosphoric acid, the acid was combined with the water and mixed for 10 minutes. The mixture with the asphalt was first mixed for 3 minutes with the proper amount of moisture; then the asphalt was added and the mixing continued for 7 more minutes.

Moisture-density tests.--Moisture-density tests were performed on each soil with no admixture and with each soil combined with the various test increments of stabilizer. All tests were performed in accordance with standard ASTM and AASHO specifications. Tests using cement as an admixture were made at 2 per cent increments up to 12 per cent on all soils except Soil III. Due to the character of this soil, tests were made at 4 per cent increments of cement up to 12 per cent. Moisture-density tests were also made on all soils with the other admixtures at the various test increments. An exception was that no moisture-density test was made with Soil III and phosphoric acid.

Molding test specimens.--Molding of all the soils and mixtures was done immediately after mixing except when RC-3 was used as an admixture. The soil and RC-3, after mixing, was allowed to stand in the open air until it had a "tacky" feeling. Molding was done with static compaction in the 2.8 inch diameter mold compacting the soil mixture to a height of 5.6 inches. With the bottom piston placed in the mold and spacers and extension attached, the properly mixed soil or soil and admixture was placed in the mold in two layers, each layer being rodded 20 strokes with the 5/8 inch rod. The amount of material placed in the mold was a predetermined weight calculated to give the maximum density, as determined from the moisture density curve, when the sample was compacted to a height of 5.6 inches. The spacers were then removed and the mold placed in alignment with the top piston which was fixed to the upper head of the loading machine. The two pistons were forced together until the dial gage indicated the proper 5.6 inch height. Loading pressure was then released, the lower piston was removed and with the mold placed on the extruding jack, the sample was extruded from the mold. After extruding, the height and weight of the sample was checked.

Each batch consisted of material for 4 samples. Two moisture content samples were taken from each batch and checked after oven drying. A tolerance of ±1 per cent was allowed in the moisture content.

In developing the test procedure other sample sizes and compaction methods were attempted but all eliminated because of certain shortcomings. Primary consideration was given to the Standard Proctor method of compaction (ASTM Method D-558-44). This method, being more or less standard for all compaction work, would be ideally suited for correlation of past tests but the sample size was unsuitable. Past studies (9) (10) have shown that a sample having a ratio of length to diameter of approximately two is necessary to overcome the effects of end restraint during the triaxial test. On the Proctor size samples with an l/d ratio of approximately one, this end restraint caused serious errors in the strength measurement. An attempt was made to trim the samples to a 1.4 inch diameter but this proved inadequate due to the scaling of the granular soil samples and the greater amount of time involved.

Another method considered was the miniature compaction equipment developed by Professor George F. Sowers in the Georgia Institute of Technology Soils Laboratory. This method consists of a 2.8 inch diameter by 6.3 inch high mold with compaction accomplished with a 5 pound hammer dropping 12 inches using 25 blows on each of 4 layers. The density obtained very closely approximates the density determined by ASTM D-559-44.

Difficulties encountered with this method occurred from the compaction planes which were formed at each layer of soil. These planes caused non-uniformity in density of the compacted samples which caused, in many cases, a low strength determination when the sample sheared along the compaction plane.

Curing.--In adopting a method of curing the test samples, primary consideration was aimed at approximating field conditions. In this respect, experience has indicated that the moisture content of a compacted highway base course will undergo very little change under normal curing conditions, i.e., unless the roadway is inundated or subjected to extreme wet or dry conditions. To approximate normal curing conditions and prevent moisture changes due to the atmospheric conditions, the test samples were placed in polyethylene freezer bags and sealed immediately after molding. The samples were then placed in a moisture room (approximately 90 per cent relative humidity and 70° F.) to prevent any variation from daily fluctuations in temperature and humidity.

Testing specimens for compressive strength.--Samples were cured for 7 and 28 days and then tested in a dry condition as they were removed from the freezer bags.

Compressive strength values of the molded samples were obtained by both the unconfined compression test and the triaxial test using a lateral confining pressure of 20 psi. Twenty-eight day samples of each soil with no admixture and with 6, 9, 12 and 15 per cent portland cement were also tested triaxially using a confining pressure of 50 psi.

After the specified curing period, the samples were removed from the sealed bags and weighed to check for any moisture changes. All samples were tested in a dry condition. Both the unconfined and confined tests were made with the sample in a standard triaxial cell of approximately 6 inches diameter and 10 inches height. In the case of the confined tests, the sample was enclosed in a thin rubber membrane and the cell sealed in order to apply the lateral pressure by compressed air.

Loading was accomplished on either a scales-type test apparatus or on a constant-strain screw type testing machine. A dial gage was placed on the loading piston and strain readings taken at various load increments. On the constant-strain test, a rate of loading of 0.05 inches of movement of the loading head per minute was used. With the test on the scales-type apparatus, loads were applied at the rate of an increment of load every 30 seconds with the increment varied to approximate 10 per cent of the ultimate load. No variations were noted from using the two types of loading equipment with the two different rates of loading. Loading was continued until the sample sheared or in the case of bulge failures, the stress-strain relationship indicated a horizontal curve. After this load was ascertained the sample was removed from the triaxial cell and a moisture sample taken for check purposes.

CHAPTER IV

EVALUATION OF TEST RESULTS

General.--Testing of the various soils and soils combined with the admixtures involved determining maximum density and optimum moisture and compressive strength. Compressive strength data of the soils and soil-portland cement mixtures were also evaluated to determine the cohesion and angle of internal friction.

Each of the five soils used in this study was combined with portland cement in increments of 2 per cent ranging from 2 to 12 per cent. Each soil was combined with a mixture of lime and flyash on a basis of 75 per cent soil and 25 per cent lime-flyash. The lime and flyash proportions in this 25 per cent was varied by ratios of lime to flyash of 1:1, 1:2, 1:5 and 1:9. The RC-3 admixture was used in percentages of 3, 5 and 7. Phosphoric acid was added in 1 and 2 per cents. All percentages of admixture were based on the dry weight of the soil.

Moisture-density.--A moisture-density curve was plotted for each soil with no admixture and for each soil with the test increments of admixture as noted above. An exception was Soil III, which was tested at 4 per cent increments of portland cement and no moisture-density tests were made on this soil with phosphoric acid as an admixture.

Tables 4 through 8 show the maximum density and optimum moisture as used in molding the samples. Values of percentages of portland cement and Soil III that were not tested were interpolated. Density and moisture

Table 4. Maximum Dry Density and Optimum Moisture for Soil I

Admixture	Maximum Dry Density	Optimum Moisture
	(lb/ft^3)	(%)
None	121.0	9.0
Cement, %		
2	122.9	10.0
4	123.0	10.5
6	123.1	11.0
8	123.9	10.8
10	123.7	10.4
12	124.9	10.5
Lime-flyash, ratio		
1:1	114.3	13.5
1:2	112.1	14.0
1:5	111.0	13.7
1:9	108.6	14.0
Phosphoric acid, %		
1	124.2	9.3
2	125.4	9.0
RC-3, %		
3	123.0	8.7
5	123.2	8.4
7	123.0	6.4

Table 5. Maximum Dry Density and Optimum Moisture for Soil II

Admixture	Maximum Dry Density	Optimum Moisture
	(lb/ft^3)	(%)
None	119.1	10.3
Cement, %		
2	120.1	11.0
4	121.9	11.0
6	122.7	10.2
8	123.1	10.6
10	123.3	10.6
12	123.9	10.1
Lime-flyash, ratio		
1:1	118.7	10.8
1:2	119.8	10.3
1:5	120.5	10.2
1:9	120.8	10.4
Phosphoric Acid, %		
1	124.9	10.5
2	125.6	9.4
RC-3, %		
3	121.9	9.5
5	121.5	8.0
7	119.1	8.5

Table 6. Maximum Dry Density and Optimum Moisture for Soil III

Admixture	Maximum Dry Density	Optimum Moisture
	(lb/ft^3)	(%)
None	101.0	9.5
Cement, %		
2	102.1	9.8
4	104.3	10.0
6	106.5	10.8
8	109.0	11.7
10	110.0	11.4
12	111.1	11.2
Lime-flyash, ratio		
1:1	114.4	11.0
1:2	112.8	11.5
1:5	109.1	12.1
1:9	108.2	13.0
Phosphoric Acid, %		
1	Used same as no Admixture	
2		
RC-3, %		
3	107.2	12.5
5	107.4	11.0
7	109.0	10.0

Table 7. Maximum Dry Density and Optimum Moisture for Soil IV

Admixture	Maximum Dry Density	Optimum Moisture
	(lb/ft^3)	(%)
None	114.2	14.6
Cement, %		
2	112.4	15.5
4	111.6	16.6
6	111.8	16.8
8	112.3	15.8
10	112.2	15.9
12	114.2	15.1
Lime-flyash, ratio		
1:1	101.0	20.0
1:2	102.1	19.8
1:5	102.0	20.0
1:9	102.0	20.0
Phosphoric Acid, %		
1	115.8	15.2
2	117.2	14.5
RC-3, %		
3	114.5	14.0
5	114.1	12.3
7	113.2	12.6

Table 8. Maximum Dry Density and Optimum Moisture for Soil V

Admixture	Maximum Dry Density	Optimum Moisture
	(lb/ft^3)	(%)
None	111.2	16.5
Cement, %		
2	111.9	16.4
4	111.9	16.7
6	111.7	17.0
8	111.2	17.3
10	111.8	16.8
12	111.2	17.3
Lime-flyash, ratio		
1:1	101.5	20.2
1:2	102.0	20.0
1:5	101.3	19.8
1:9	101.4	19.7
Phosphoric Acid, %		
1	115.8	15.9
2	117.4	14.7
RC-3, %		
3	113.3	13.5
5	112.3	13.7
7	111.5	13.7

values used for Soil III and phosphoric acid were the same as for no admixture in that soil. Curves showing the variation in maximum dry density and optimum moisture versus admixture are shown in Figures 6 through 10.

For Soil I, the addition of portland cement produced an increase in maximum density with increasing amounts of cement while the optimum moisture increased slightly with the addition of cement then remained nearly the same as the cement percentage increased. Maximum density with phosphoric acid increased for this soil with no change in moisture. Asphalt caused an increase in density which was almost constant with the higher percentages while the moisture dropped with increasing amounts of RC-3. The addition of lime and flyash to this soil caused a marked reduction at the higher lime-flyash ratios. Optimum moisture increased with the addition of lime-flyash, then remained nearly the same as the lime-flyash ratio increased.

Evaluation of Figure 7 for Soil II shows that increasing percentages of cement causes increasing density with little change in optimum moisture. The addition of phosphoric acid caused a substantial increase in density while the higher per cent of acid increased only slightly over the lesser per cent. No appreciable change occurred in the optimum moisture. Addition of RC-3 to this soil increased the density at 3 and 5 per cent while at 7 per cent, the density dropped to the same value as the original soil. Optimum moisture decreased with the RC with the greatest decrease at 5 per cent. Adding lime and flyash to this soil produced a very slight decrease in density at the lowest ratio of lime-flyash with

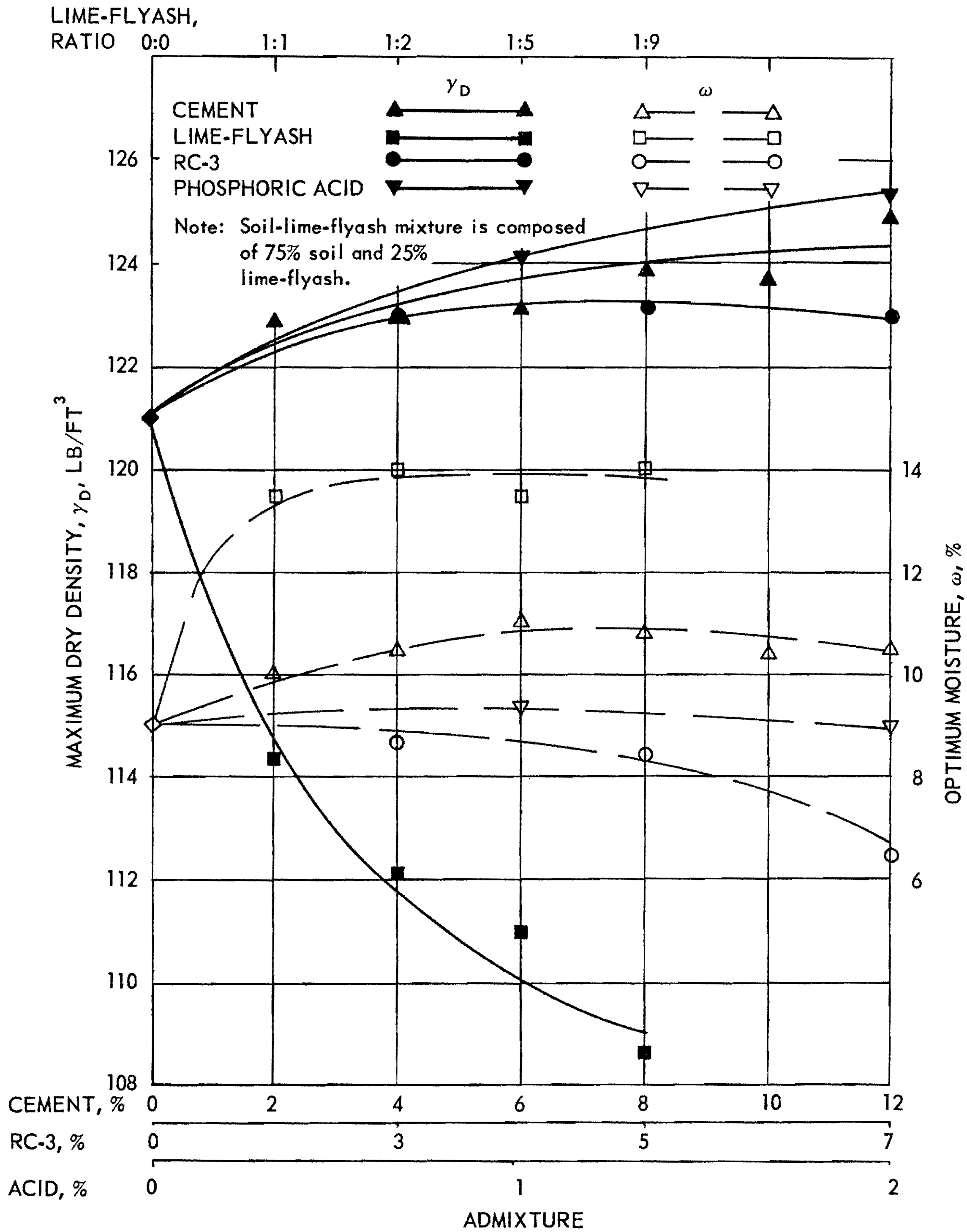


Figure 6. Relationship of Maximum Dry Density and Optimum Moisture Versus Admixture for Soil I.

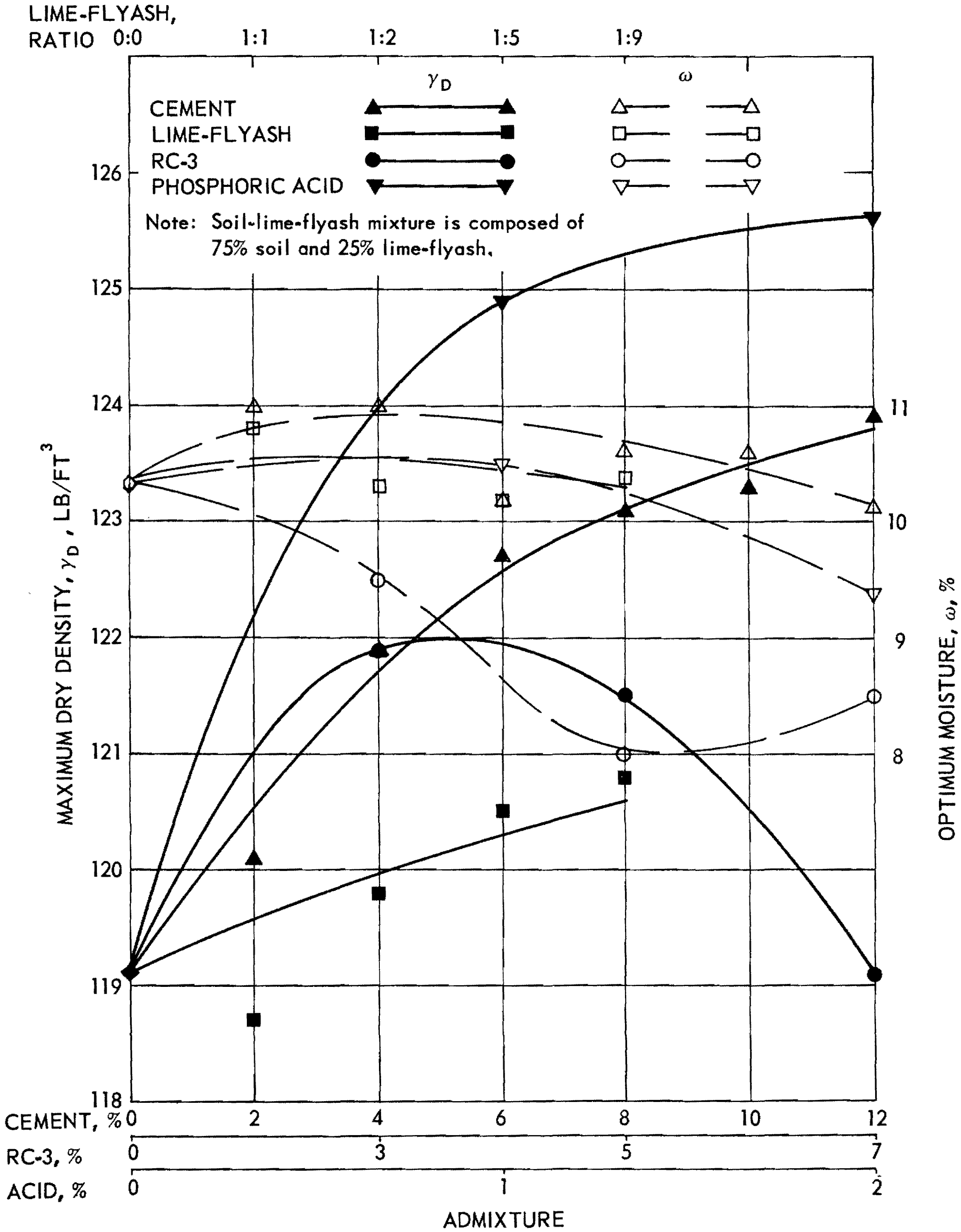


Figure 7. Relationship of Maximum Dry Density and Optimum Moisture Versus Admixture for Soil II.

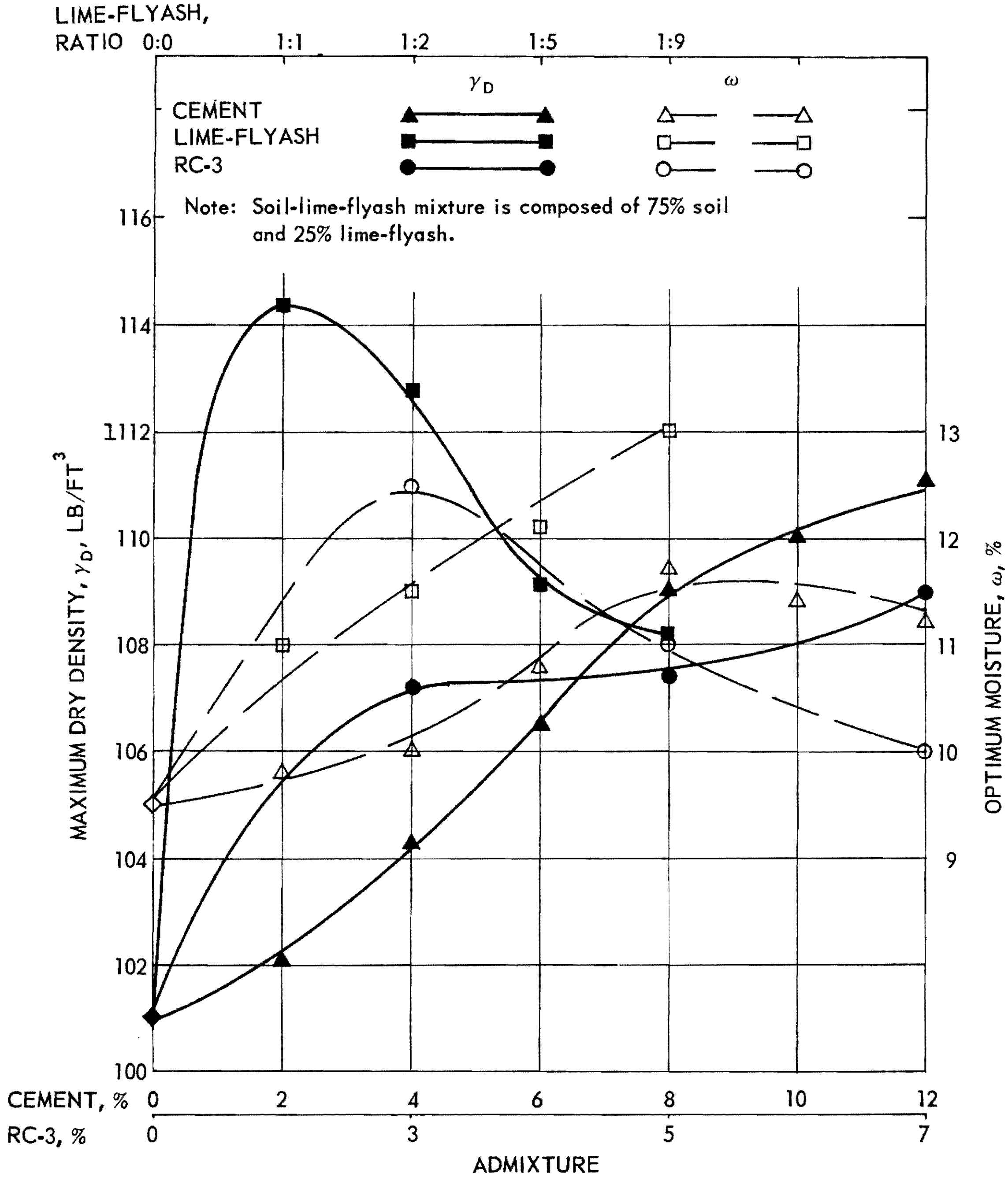


Figure 8. Relationship of Maximum Dry Density and Optimum Moisture Versus Admixture for Soil III.

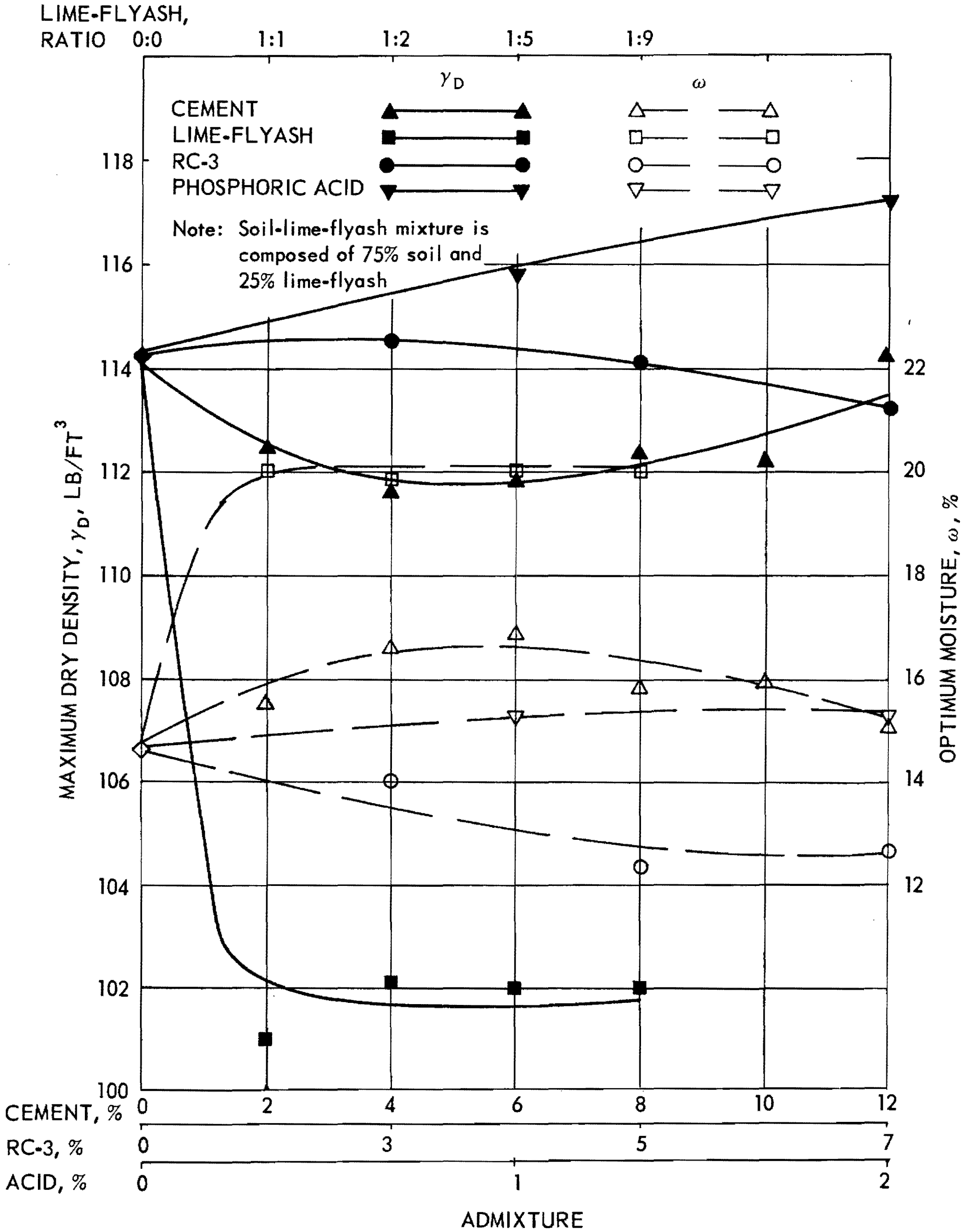


Figure 9. Relationship of Maximum Dry Density and Optimum Moisture Versus Admixture for Soil IV.

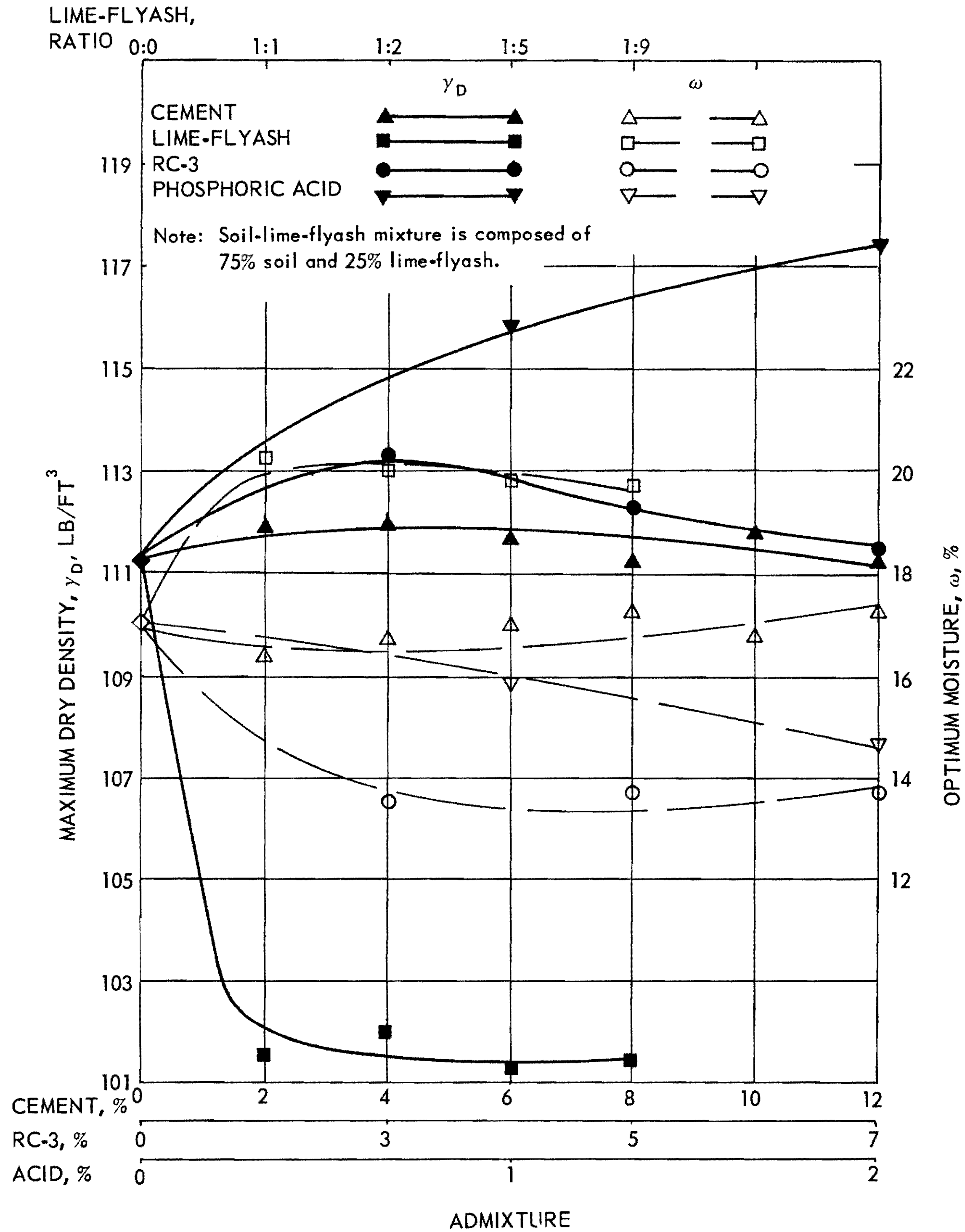


Figure 10. Relationship of Maximum Dry Density and Optimum Moisture Versus Admixture for Soil V.

the density increasing slightly with the higher ratios. Again, optimum moisture was changed only slightly.

Soil III with its fine uniform grains was very difficult to compact as the compaction hammer sheared the soil in the mold. Optimum moisture in this soil was not critical as the compaction could be accomplished over a fairly wide range of moisture contents. The addition of an admixture improved the compaction characteristics. Portland cement as an admixture caused a nearly linear increase in density with increase in percentage of cement. Optimum moisture also increased but to a lesser amount at the higher cement contents. RC-3 produced an increase in density but little change with increase in RC-3 per cent. Optimum moisture increased at 3 per cent RC but then dropped to approximately the original soil value for 7 per cent RC. The addition of the smallest proportion of lime-flyash gave a marked increase in density with a lesser increase as the lime-flyash ratio increased. Optimum moisture for this mixture increased approximately linear with increasing amounts of flyash.

The addition of admixtures to Soil IV had only slight effect on density except the admixture, lime-flyash. Portland cement added to this soil caused a slight reduction in density, the reduction becoming less at the higher percentages. Optimum moisture increased slightly with the intermediate percentages of cement with practically the same moisture content at higher percentages as with the original soil. A small linear increase in density with phosphoric acid was noted with no change in moisture. The only change with this soil and asphalt was a slight decrease in density at 7 per cent and small reduction in moisture with increasing asphalt percentages. With the addition of lime-flyash, a marked decrease in density was noted with a corresponding rise in optimum moisture. Varying the ratio of lime to flyash did not effect this drop in density or rise in moisture.

For Soil V, the addition of portland cement had no effect on density or moisture. Phosphoric acid produced an increase in density with increasing percentages of admixture while the optimum moisture had a corresponding decrease. RC-3 increased the density slightly at 3 and 5 per cent with no change at 7 per cent. Optimum moisture decreased with the RC but no change occurred with varying percentages. Lime-flyash caused a marked reduction in density and a corresponding increase in moisture but the density and moisture values remained nearly constant with varying ratios of lime to flyash.

Compressive strength.--In evaluating the molded samples, only samples molded within 1 per cent of optimum moisture were used. For each test, 4 samples were molded for unconfined compression at 7 and 28 days and 4 samples for triaxial testing at 7 and 28 days. For compressive strength evaluation, only the values which were within 10 per cent of the average of the other samples were used. In most instances, the results were consistent and represent the average of 4 samples tested. Compressive strength results are shown in Tables 9 through 13. Figures 11 through 20 show curves of compressive strength versus admixture for the confined triaxial tests and for the unconfined tests.

As shown in Figures 11 and 12 and Table 9 the compressive strength of Soil I was increased with the addition of admixtures. Portland cement was, by far, the most beneficial admixture with a small gain in strength with low percentages of cement and greater increases in strength with the

Table 9. Compressive Strength for Soil I

Admixture	Compressive Strength, psi 7 Day 0*	7 Day 20	28 Day 0	28 Day 20
None	15	81	7	81
Cement, %				
2	20	114	23	139
4	50	163	81	239
6	98	254	148	261
8	247	354	274	516
10	431	580	557	693
12	572	726	712	883
Lime-flyash, ratio				
1:1	48	133	57	159
1:2	41	138	36	115
1:5	58	156	74	180
1:9	16	96	25	110
Phosphoric acid, %				
1	14	126	27	133
2	18	95	29	120
RC-3, %				
3	14	53	22	92
5	19	67	35	89
7	26	71	44	90

*Note: The 0 and 20 indicate confining pressure in psi in the triaxial test.

Table 10. Compressive Strength for Soil II

Admixture	Compressive Strength, psi 7 day 0*	7 day 20	28 day 0	28 day 20
None	15	85	13	84
Cement, %				
2	95	189	144	228
4	217	317	220	346
6	378	439	354	488
8	475	565	667	728
10	618	717	880	945
12	769	864	1035	1114
Lime-flyash, ratio				
1:1	128	232	153	277
1:2	115	229	140	266
1:5	88	197	100	224
1:9	72	191	85	205
Phosphoric acid, %				
1	34	126	41	135
2	34	120	47	143
RC-3, %				
3	15	63	16	66
5	30	75	29	70
7	24	53	25	51

*Note: The 0 and 20 indicate confining pressure in psi in the triaxial test.

Table 11. Compressive Strength for Soil III

Admixture	Compressive Strength, psi 7 day		28 day	
	0*	20	0	20
None	0	0	0	59
Cement, %				
2	0	61	3	58
4	3	68	3	62
6	7	89	5	68
8	49	163	128	216
10	122	240	222	329
12	262	324	389	484
Lime-flyash, ratio				
1:1	21	142	14+	106+
1:2	7	108	12+	62+
1:5	9	94	11+	89+
1:9	8+	70+	8+	78+
Phosphoric acid, %				
1	2	70	2	70
2	1	68	2	70
RC-3, %				
3	2	52	2	54
5	2	60	3	66
7	3	69	5	72

*Note: The 0 and 20 indicate confining pressure in psi in the triaxial test.

+These samples were molded with Columbia, S. C. flyash.

Table 12. Compressive Strength for Soil IV

Admixture	Compressive Strength, psi 7 day		28 day	
	0*	20	0	20
None	33	58	38	59
Cement, %				
2	104	142	109	167
4	249	291	274	347
6	272	344	369	465
8	289	367	376	482
10	349	466	458	568
12	457	469	550	640
Lime-flyash, ratio				
1:1	53	128	65	149
1:2	45	116	51	122
1:5	43	108	51	143
1:9	42	108	49	128
Phosphoric acid, %				
1	41	103	46	117
2	78	149	115	182
RC-3, %				
3	34	51	37	57
5	37	54	44	63
7	42	48	44	46

*Note: The 0 and 20 indicate confining pressure in psi in the triaxial test.

Table 13. Compressive Strength for Soil V

Admixture	Compressive Strength, psi 7 day 0*	7 day 20	28 day 0	28 day 20
None	23	38	25	45
Cement, %				
2	73	123	81	133
4	188	245	246	290
6	209	282	291	352
8	268	339	341	417
10	296	382	394	457
12	283	367	413	504
Lime-flyash, ratio				
1:1	48	130	113	199
1:2	44	126	91	177
1:5	34	106	75	152
1:9	37	100	65	143
Phosphoric acid, %				
1	46	77	63	108
2	58	100	82	129
RC-3, %				
3	32	55	33	67
5	31	57	35	68
7	40	65	32	56

*Note: The 0 and 20 indicate confining pressure in psi in the triaxial test.

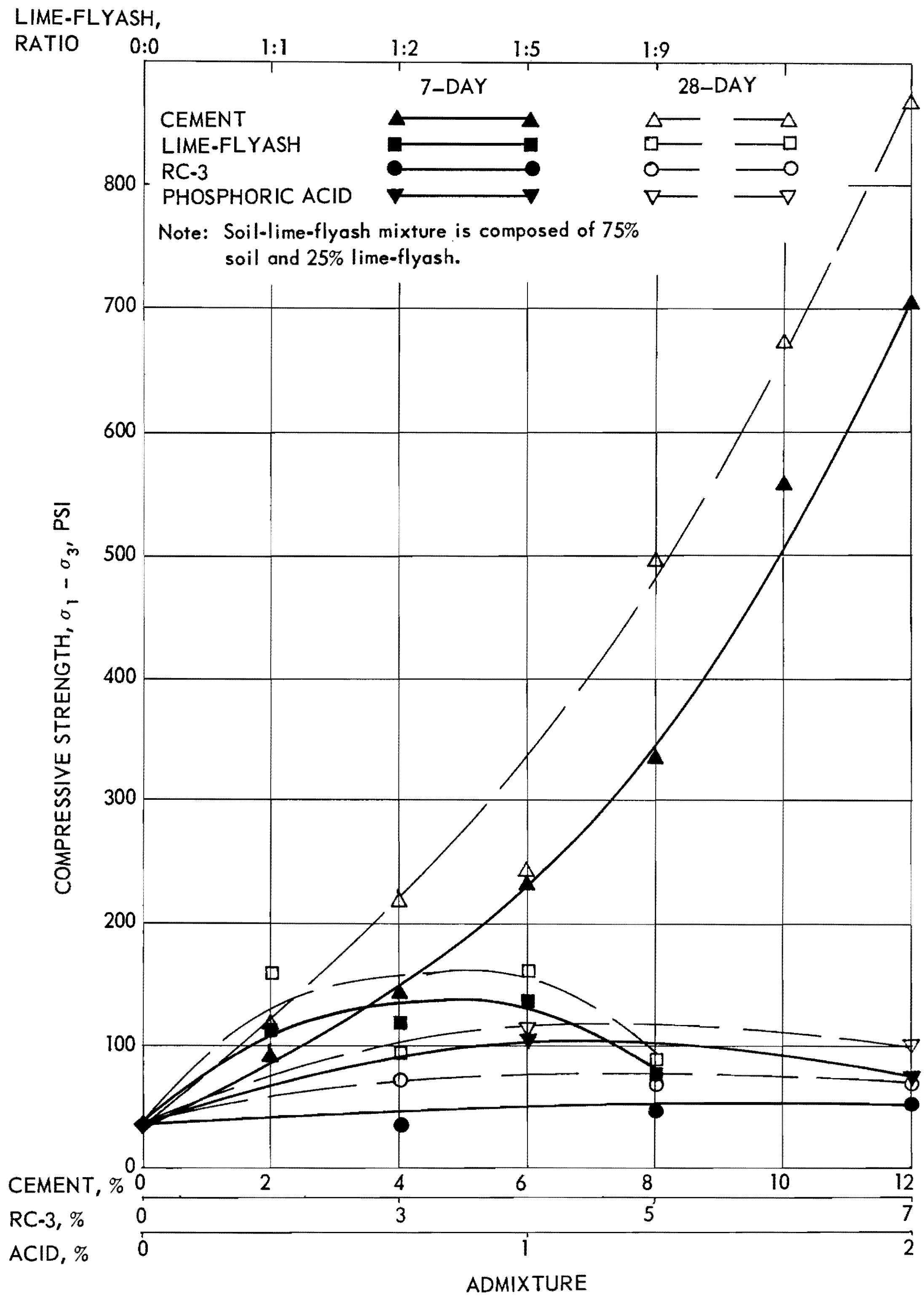


Figure 11. Relationship of Confined Compressive Strength and Admixture for Soil I.

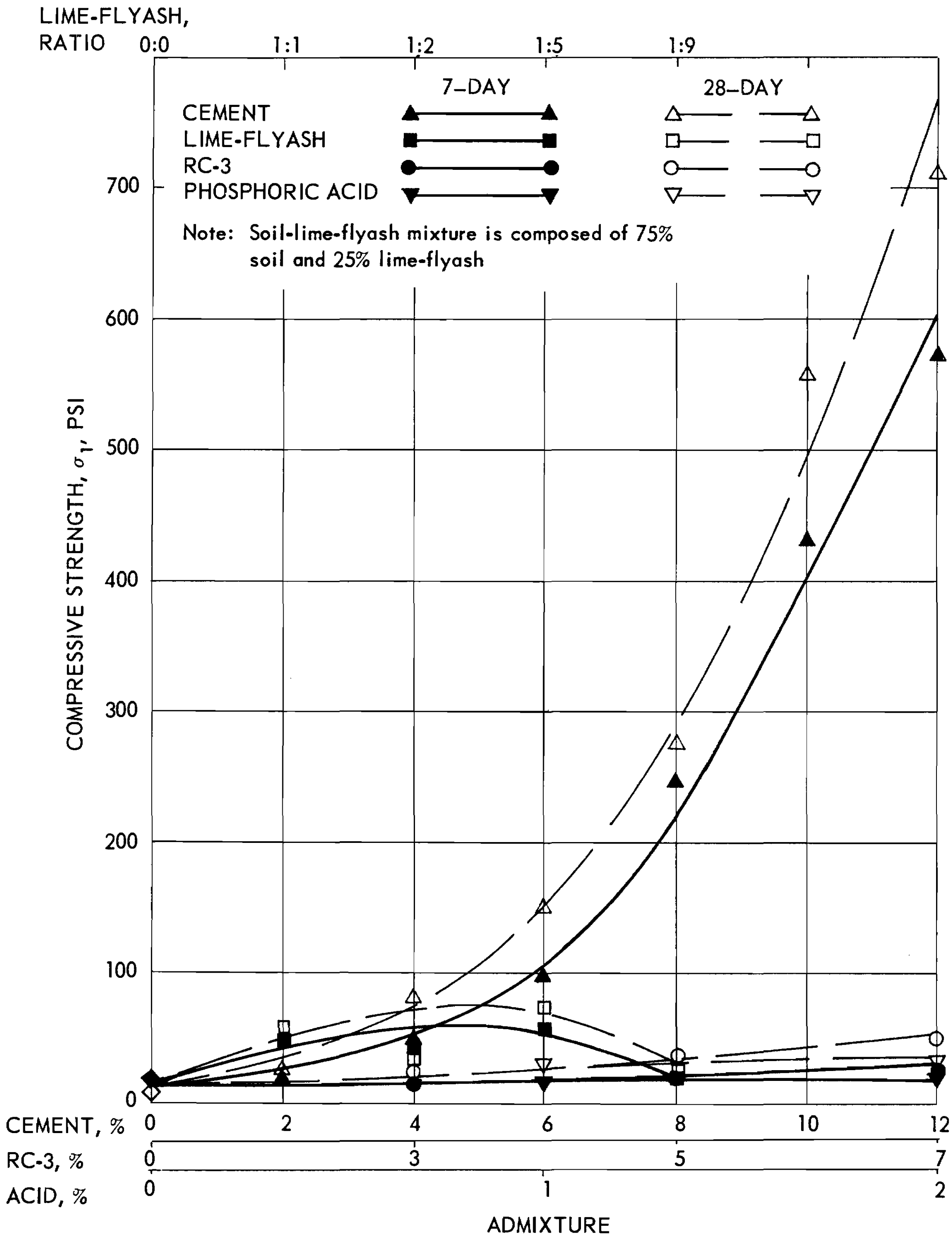


Figure 12. Relationship of Unconfined Compressive Strength and Admixture for Soil I.

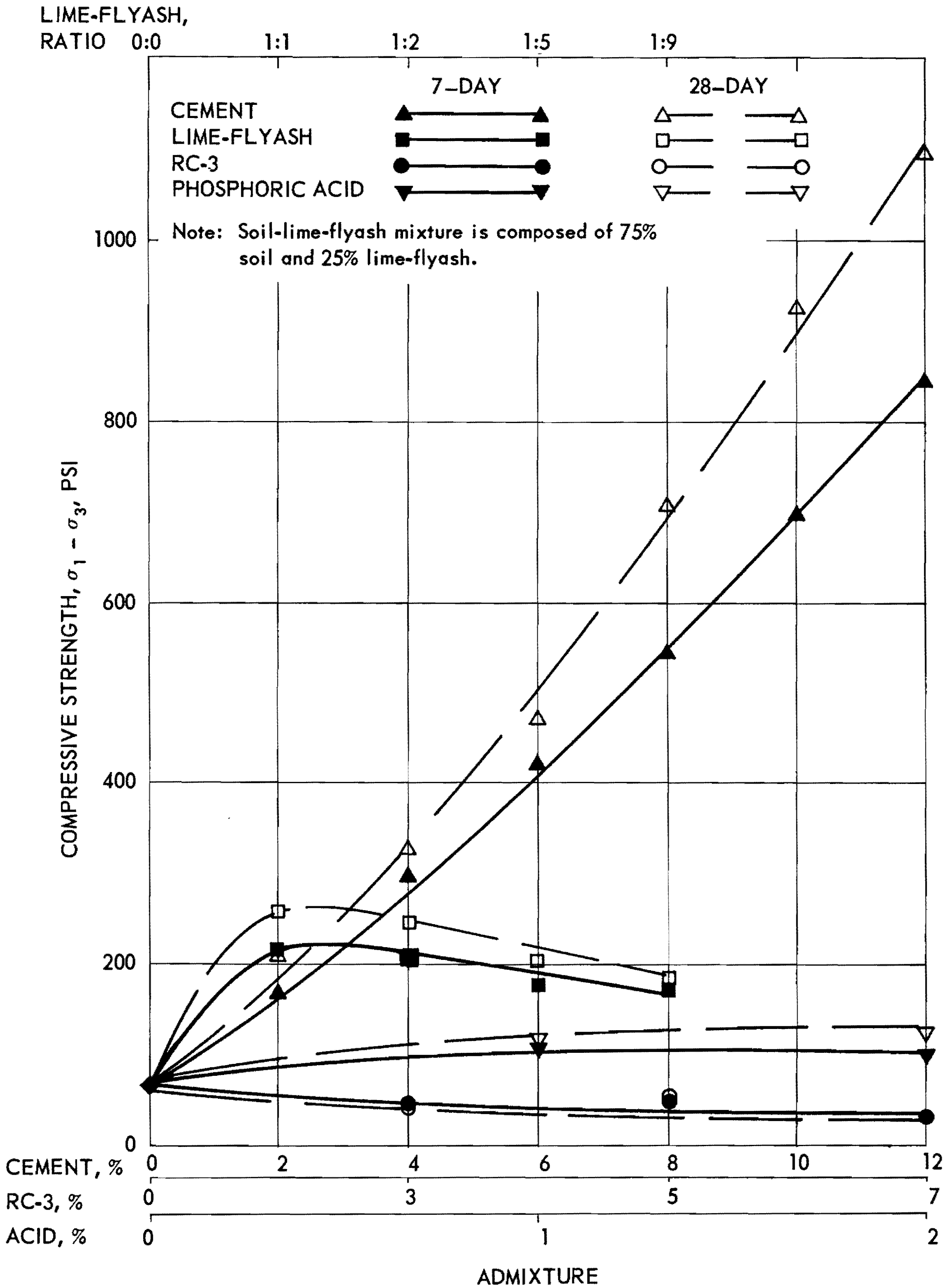


Figure 13. Relationship of Confined Compressive Strength and Admixture for Soil II.

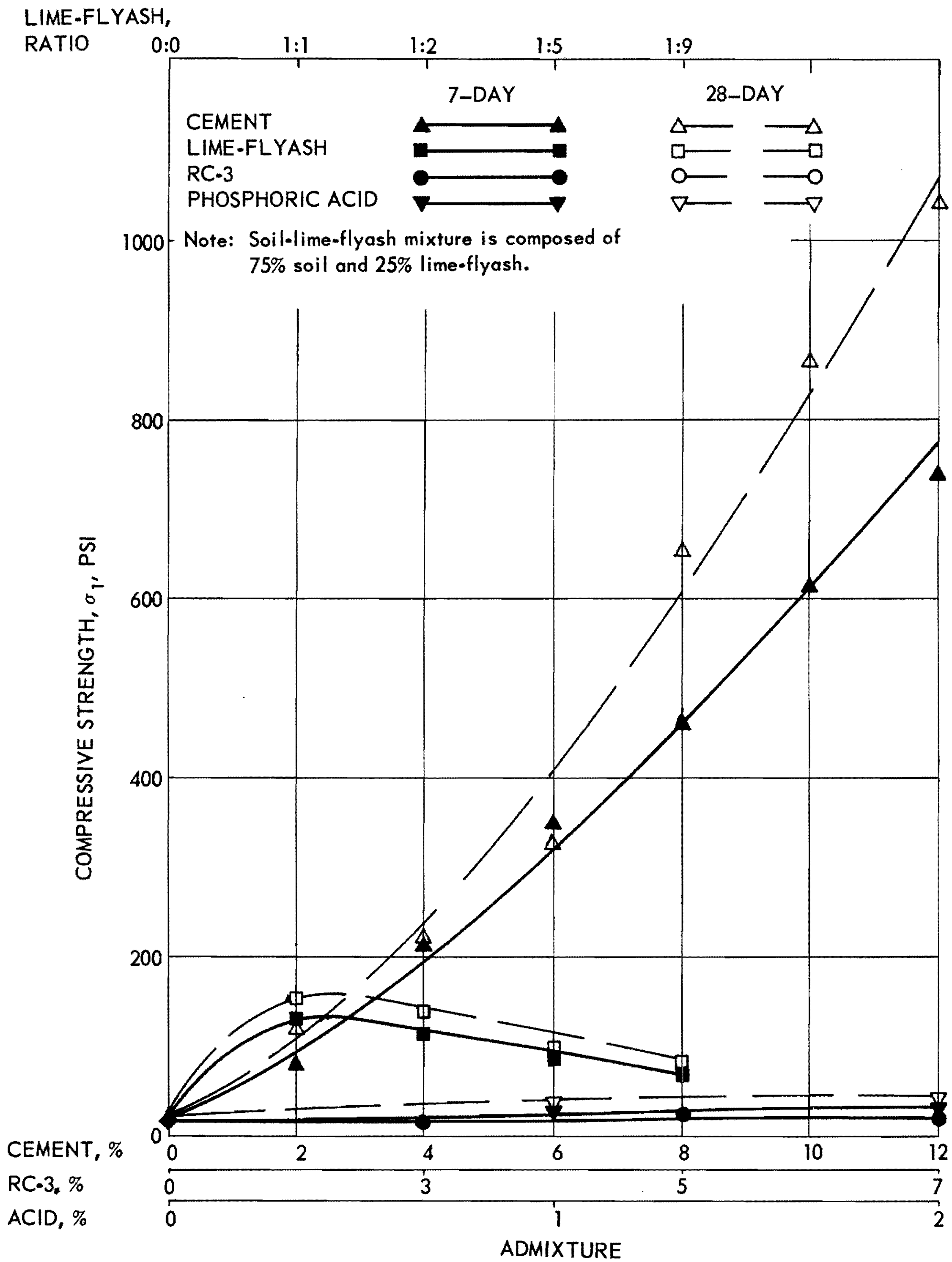


Figure 14. Relationship of Unconfined Compressive Strength and Admixture for Soil II.

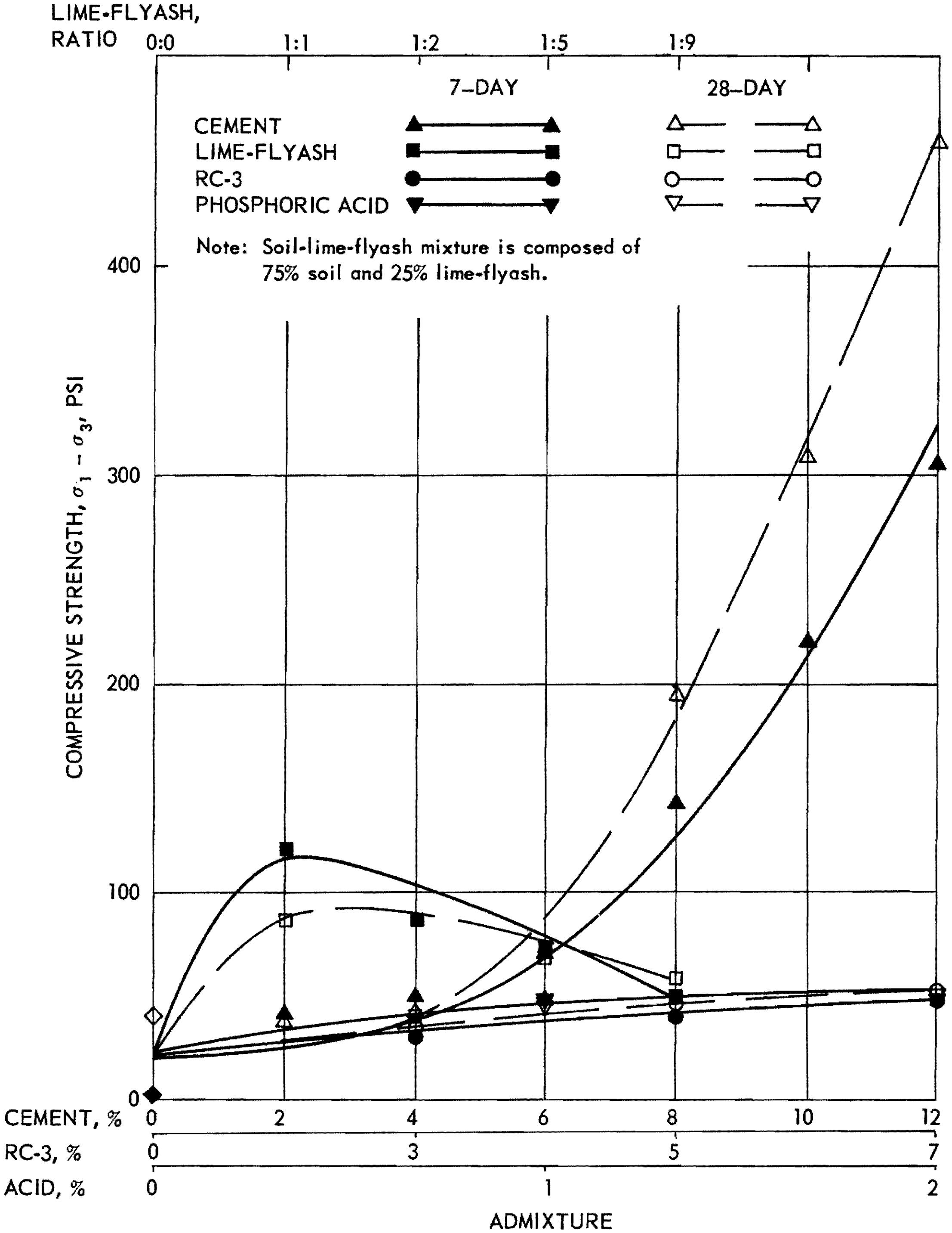


Figure 15. Relationship of Confined Compressive Strength and Admixture for Soil III.

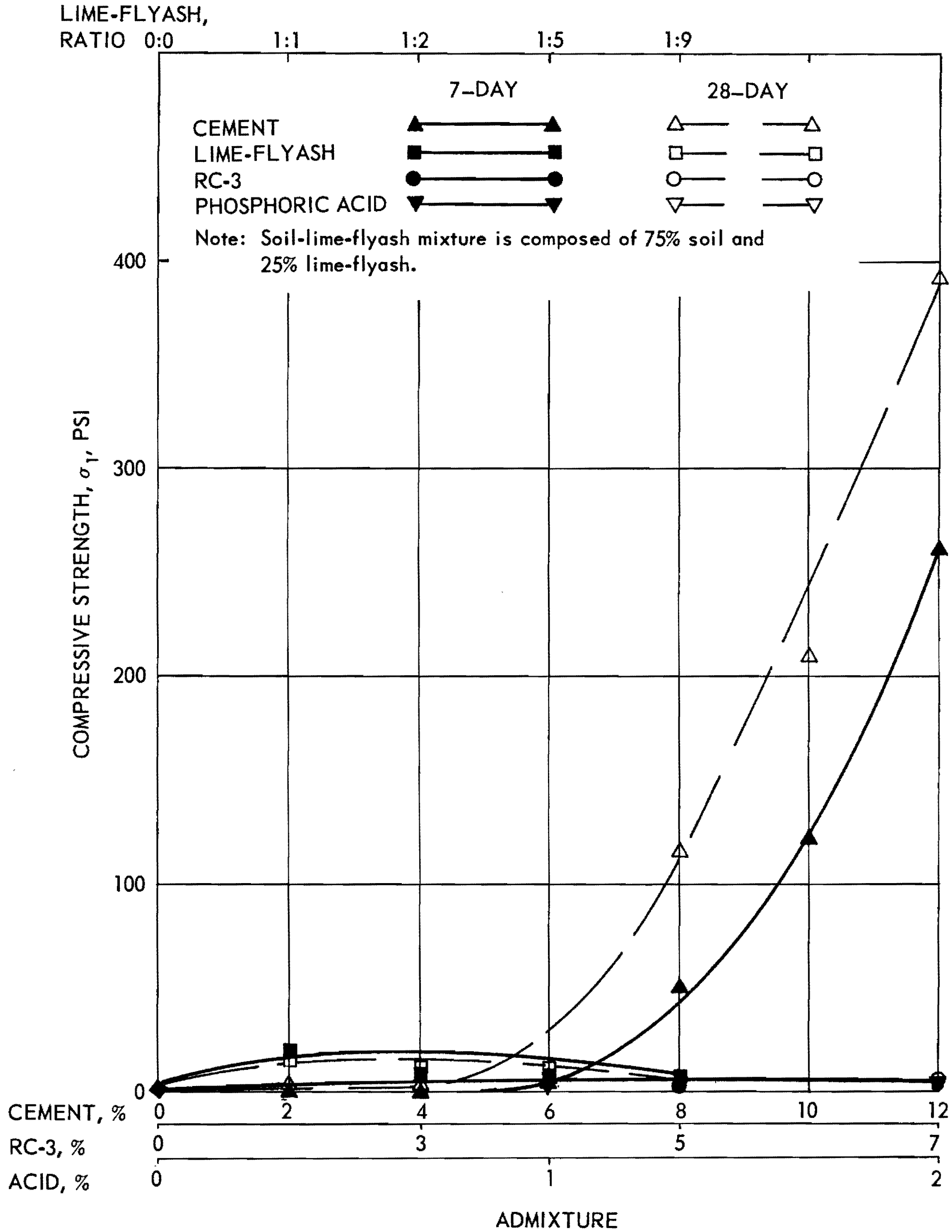


Figure 16. Relationship of Unconfined Compressive Strength and Admixture for Soil III.

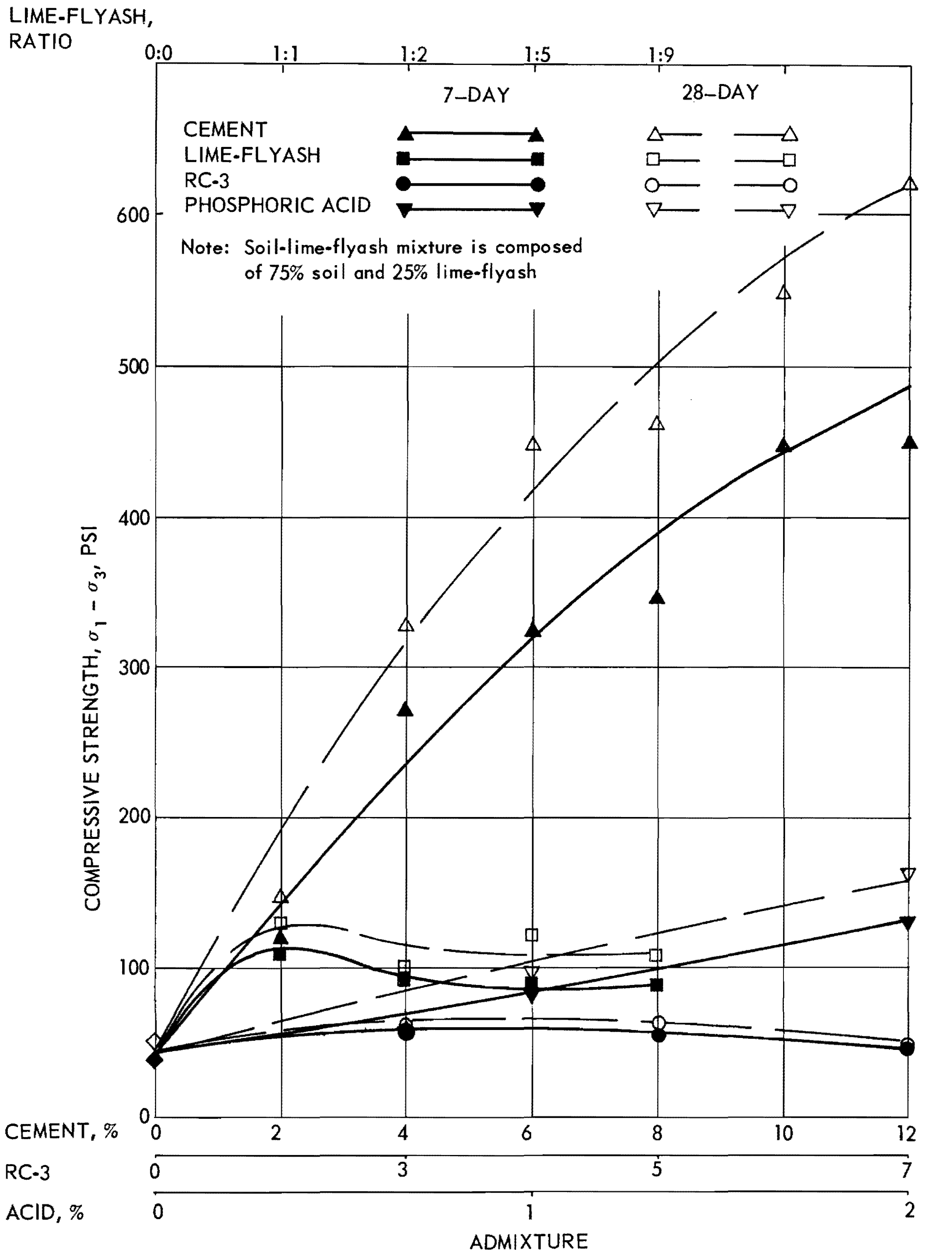


Figure 17. Relationship of Confined Compressive Strength and Admixture for Soil IV.

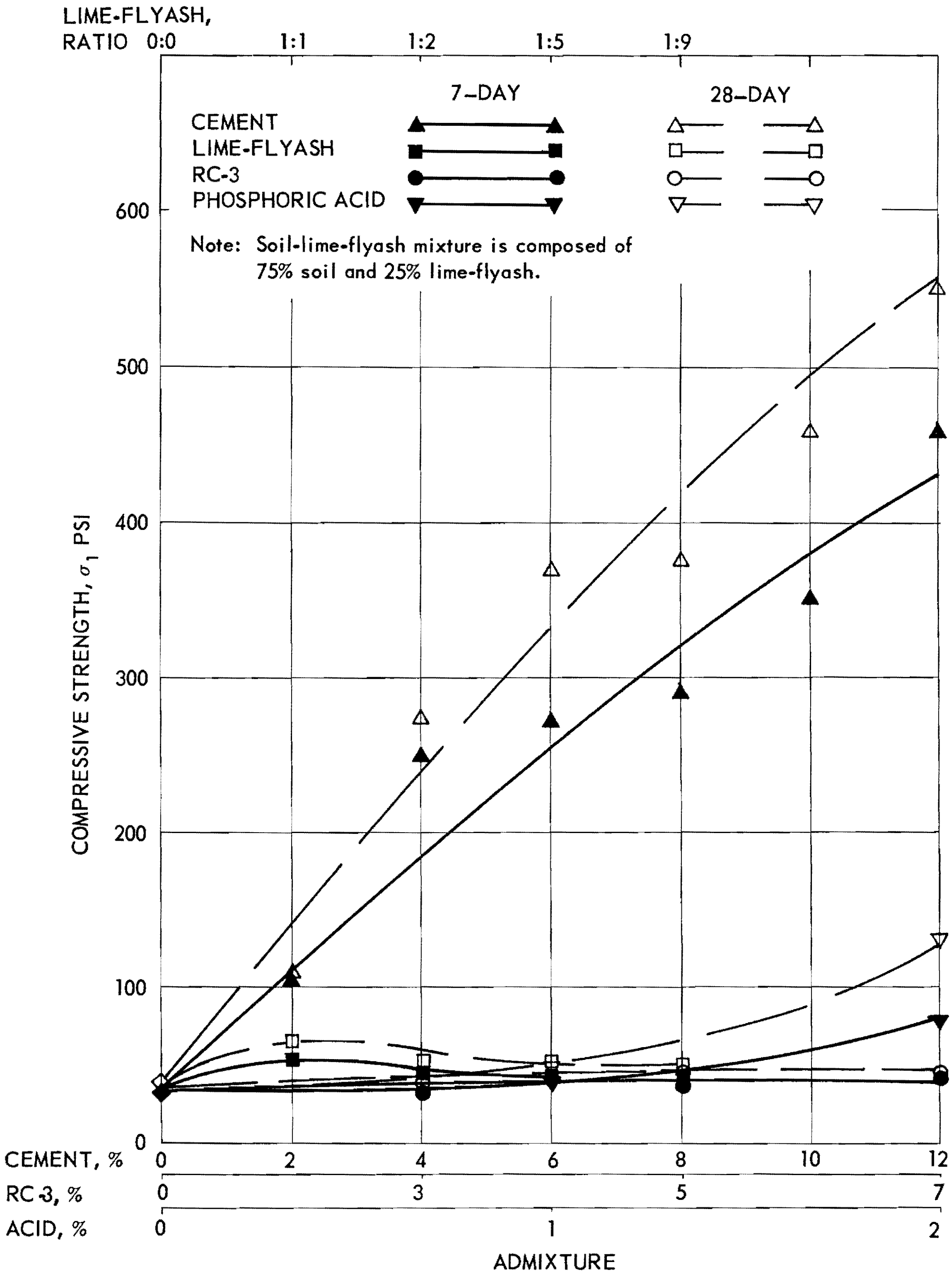


Figure 18. Relationship of Unconfined Compressive Strength and Admixture for Soil IV.

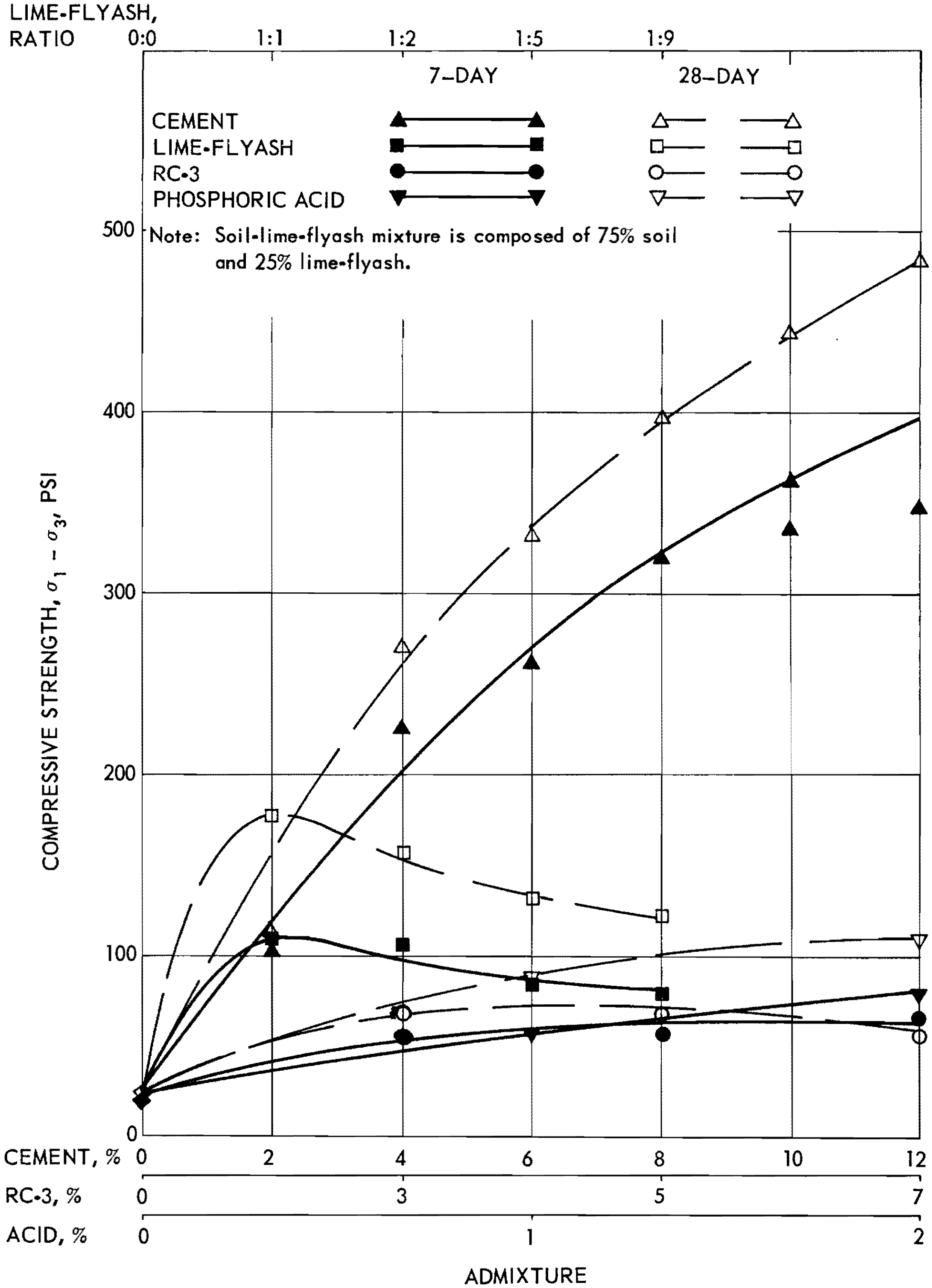


Figure 19. Relationship of Confined Compressive Strength and Admixture for Soil V.

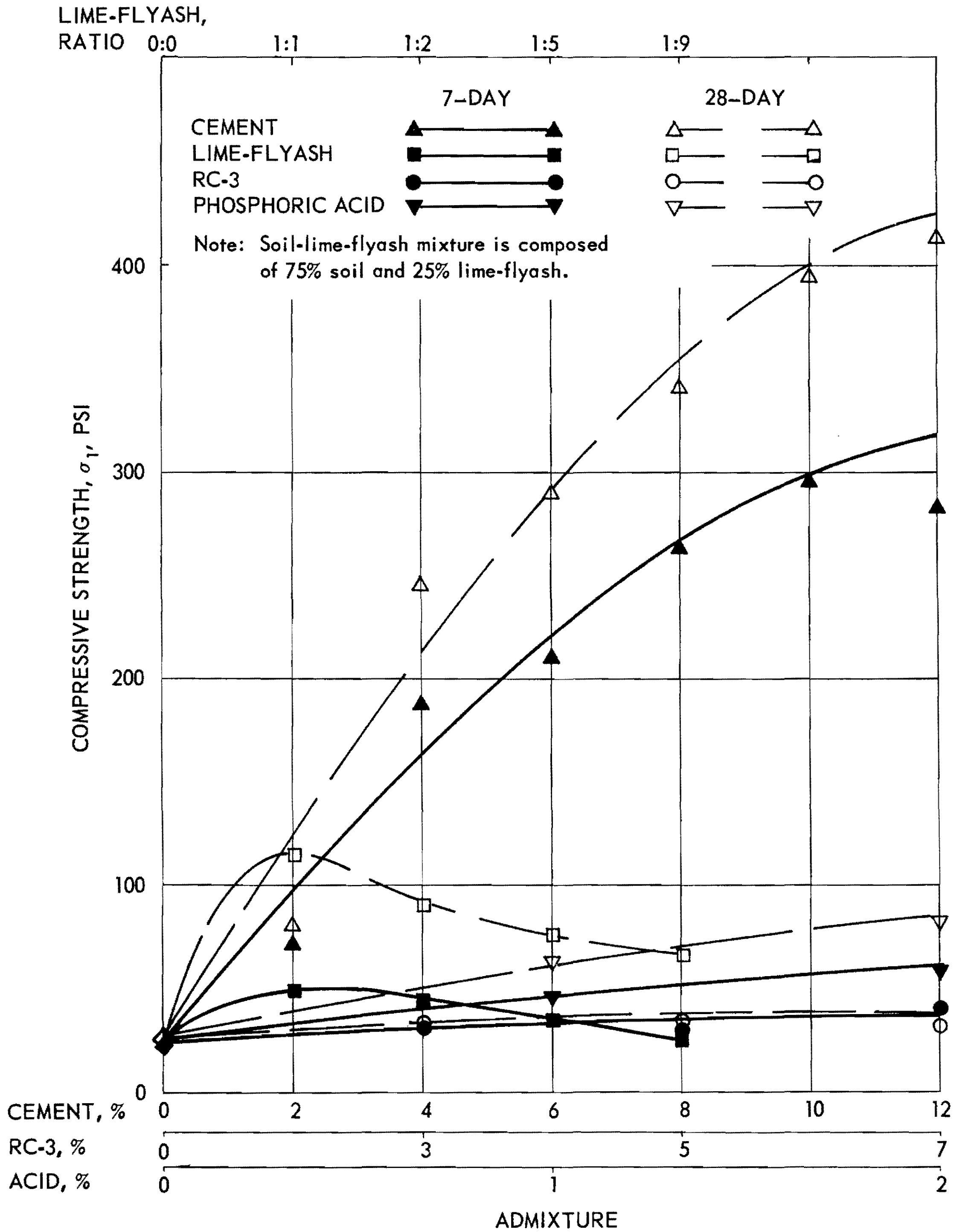


Figure 20. Relationship of Unconfined Compressive Strength and Admixture for Soil V.

higher percentages. The triaxial test curves and unconfined test curves had approximately the same shape with the triaxial test curves showing greater improvement at the lower percentages of cement. The 28 day test curves showed approximately a constant increase in strength over the 7 day test curves from 6 per cent to 12 per cent cement. The lime-flyash mixture gave an improved strength with little variation with the change in ratio of lime to flyash except at the 1:9 ratio which had a drop in strength to slightly higher than the soil with no admixture. Maximum strength obtained with this admixture compared with approximately 3 to 4 per cent portland cement. Strength gain with phosphoric acid was about double the raw soil strength but the maximum strength was only approximately 100 and 50 psi for the confined and unconfined tests, respectively. The addition of RC-3 caused only a slight increase in strength and was the poorest admixture from a strength standpoint.

The variation in strength with the addition of admixtures to Soil II is shown in Figures 13 and 14 and tabulated in Table 10. Portland cement was the most beneficial admixture with approximately a linear increase in strength with increasing percentages of cement. A strength of over 1000 psi was obtained with 12 per cent cement in both the triaxial and unconfined tests. There was greater increase in the 28 day strengths over the 7 day strength with increasing amounts of cement. The lime-flyash admixture caused an increase in strength with the maximum strength obtained with a 1:1 lime-flyash ratio. This maximum strength compared with the strength of approximately 3 per cent portland cement. A negligible increase in strength was obtained with phosphoric acid. The addition of RC-3 caused a decrease in strength with the greatest decrease at 3 per cent.

From Figures 15 and 16 and Table 11, it is noted that the addition of portland cement to Soil III had negligible effect on strength up to 6 per cent. The addition of more than 6 per cent greatly increased the strength with a rapid rise in the strength curves up through 12 per cent. The increase in strength of the 28 day tests also was greater at the higher cement contents. The lime-flyash mixture improved the strength of this soil with the 1:1 lime-flyash showing the greatest improvement. The strength increase caused by lime-flyash compared to approximately 7 per cent portland cement. The decrease in 28 day strength as compared to the 7 day strength may be attributed to the use of flyash from different plants for these tests. Phosphoric acid and RC-3 gave negligible increases in strength with this soil.

The addition of portland cement to Soil IV as shown in Figures 17 and 18 and Table 12 caused a marked increase in strength even with the 2 per cent addition. This increase in strength was approximately linear with the 28 day strength increasing over the 7 day strength at higher cement percentages. An increase in strength of approximately 300 per cent at 28 days was noted with the addition of 2 per cent phosphoric acid. This was approximately the same increase effected by 2 to 3 per cent portland cement. The addition of 1 per cent acid was less effective as was the 7 day curing period. The lime-flyash mixture was slightly effective at the 1:1 lime-flyash ratio with less strength gains at the higher flyash contents. Lime-flyash was more effective when tested in the triaxial test. RC-3 was not effective in increasing the strength of this soil, even reducing the strength with 7 per cent asphalt.

Figures 19 and 20 and Table 13 show the variation in strength with the various admixtures and Soil V. Portland cement was the most effective stabilizer with approximately 1000 per cent increase in strength at 12 per cent cement. The rate of increase was greatest up to 8 per cent. A steady increase in 28 day strength over the 7 day strength was noted with increasing amounts of cement. The lime-flyash mixture was also an effective stabilizer in this soil, especially after the 28 day curing period. Strengths approximately 4 times greater than the raw soil was effected by the addition of a 1:1 lime-flyash ratio with slightly less strength gains with the higher flyash content. A strength increase up to approximately 300 per cent was obtained with 2 per cent phosphoric acid with slightly less increase at 1 per cent. A negligible increase was obtained with RC-3 with little variation using the different percentages.

Cohesion and internal friction.--The load carrying ability of a soil is determined by its "cohesion" and/or "internal friction." In a sandy soil the mechanical interlocking of the solid particles provide the strength while in a cohesive soil, the mutual attraction between particles, which involves forces of electro-chemical nature, provide resistance to displacement. In most soils, the load carrying properties are derived from a combination of "cohesion" and "internal friction." These two parameters may be easily determined from plotting graphically the results of triaxial tests. This graphical plot is called a Mohr's diagram. Points which represent σ_1 and σ_3, compressive normal stresses and confining stresses respectively, are plotted along the abscissa and joined by a circle whose center is also on the abscissa. Circles are drawn corresponding to various confining pressures and a tangent is drawn to the circles. The intercept of this tangent with the y-axis is called "cohesion" and the slope of the tangent in degrees is the angle of "internal friction."

Data from the Mohr's diagram is tabulated in Table 14 and the variation in cohesion and angle of internal friction versus per cent portland cement is shown in Figures 21 through 25. Tests were made with the five soils combined with 0, 6, 9, 12 and 15 per cent portland cement using confining pressures of 0, 20 and 50 psi. Individual Mohr's diagrams for each per cent cement are shown in the appendix in Figures 26 through 50.

The addition of portland cement caused an increase in cohesion and angle of internal friction in all soils tested. Soil I, which had a cohesion of 2 psi, and angle of internal friction of 33 degrees with no admixture, had an increase in angle of internal friction at 6 per cent cement to approximately 50 degrees and remained constant with increasing amounts of cement. The cohesion in this soil increased rapidly up to about 9 per cent cement where the rate of increase decreased but with an increase through 15 per cent cement.

The angle of internal friction of Soil II was 29° with no admixture and increased to approximately 45° at 6 per cent cement where it remained approximately constant with increasing amounts of cement. Cohesion in this soil with no admixture was 5 psi and a marked increase was noted up to 12 per cent cement where the rate of increase lessened and at 15 per cent cement a cohesion of 275 psi was obtained.

Soil III, a fine uniformly graded sand, had no cohesion and an angle of internal friction of 29° with no admixture. The addition of

Table 14. Mohr's Diagram Data

Soil No.	Cement Content	Compressive Strength			Cohesion, C	Angle of Internal friction, ø
	%	psi			psi	degrees
		0*	20	50		
I	0	7	81	179	2	33
	6	148	261	574	30	49
	9	564	774	862	110	49
	12	712	883	1113	130	51
	15	750	1043	1080	145	49
II	0	13	84	158	5	29
	6	354	488	702	70	48
	9	751	873	1004	162	43
	12	1035	1114	1271	239	41
	15	1310	1448	1589	275	45
III	0	0	59	142	0	29
	6	5	68	165	2	34
	9	210	305	413	52	38
	12	389	484	772	88	41
	15	793	884	1054	180	41
IV	0	38	59	86	19	0
	6	369	465	564	95	36
	9	415	466	569	120	30
	12	550	640	836	136	40
	15	647	744	862	153	39
V	0	25	45	73	12	0
	6	291	352	440	83	31
	9	403	492	551	115	31
	12	413	504	585	123	31
	15	454	608	672	136	31

*Note: The 0, 20, and 50 indicate confining pressure in psi in the triaxial test.

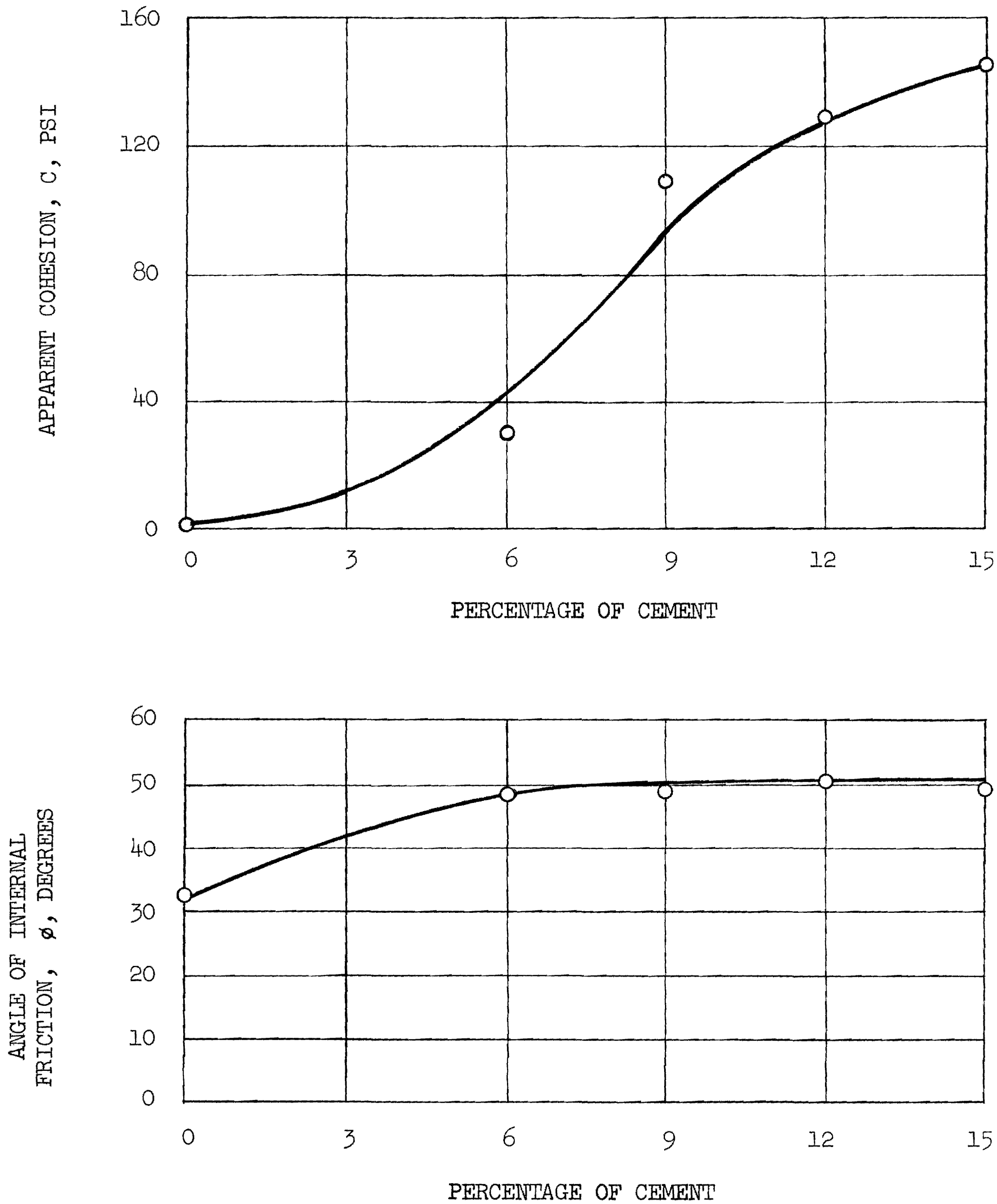


Figure 21. Apparent Cohesion and Angle of Internal Friction Versus Cement Content for Soil I.

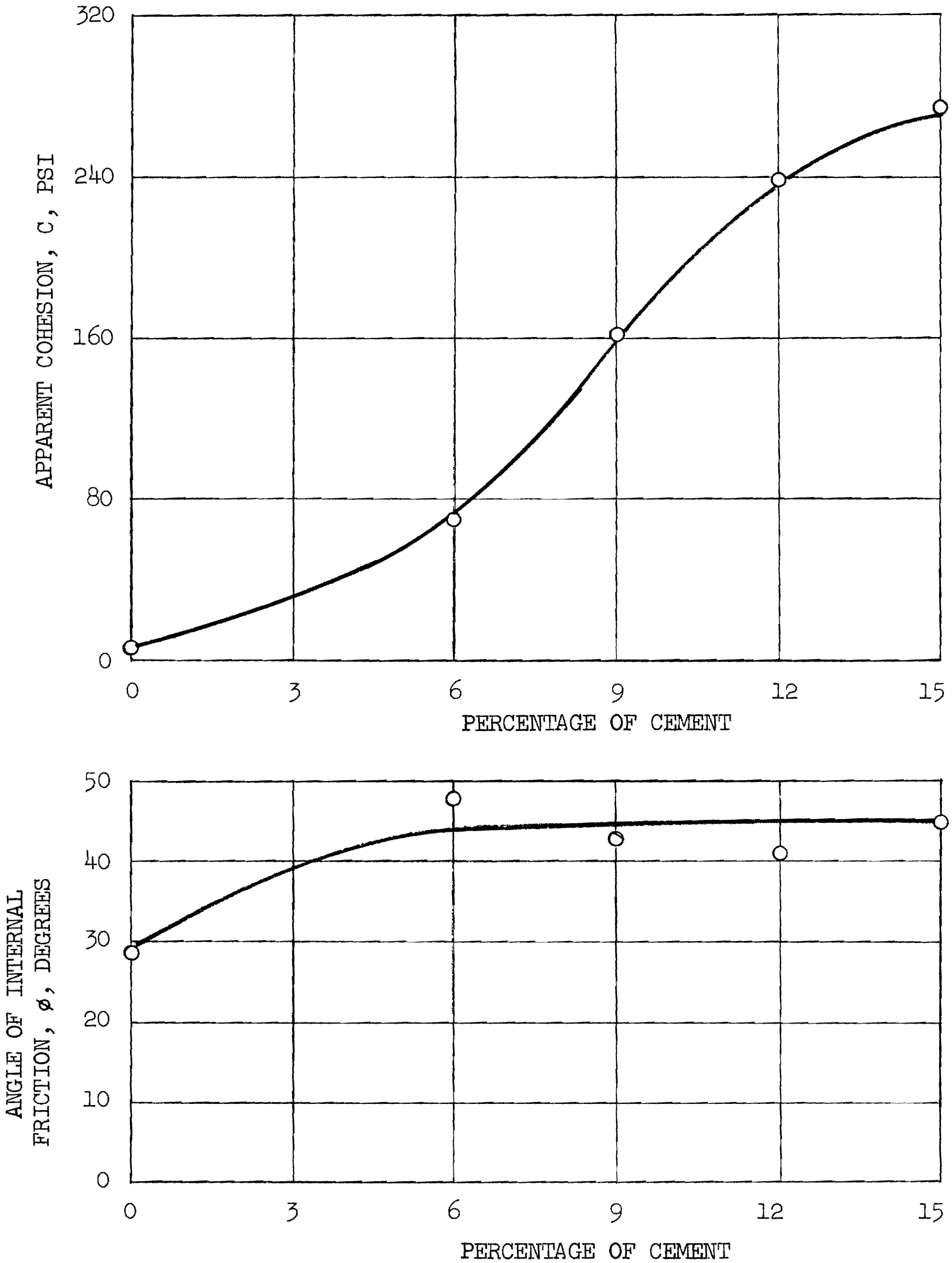


Figure 22. Apparent Cohesion and Angle of Internal Friction Versus Cement Content for Soil II.

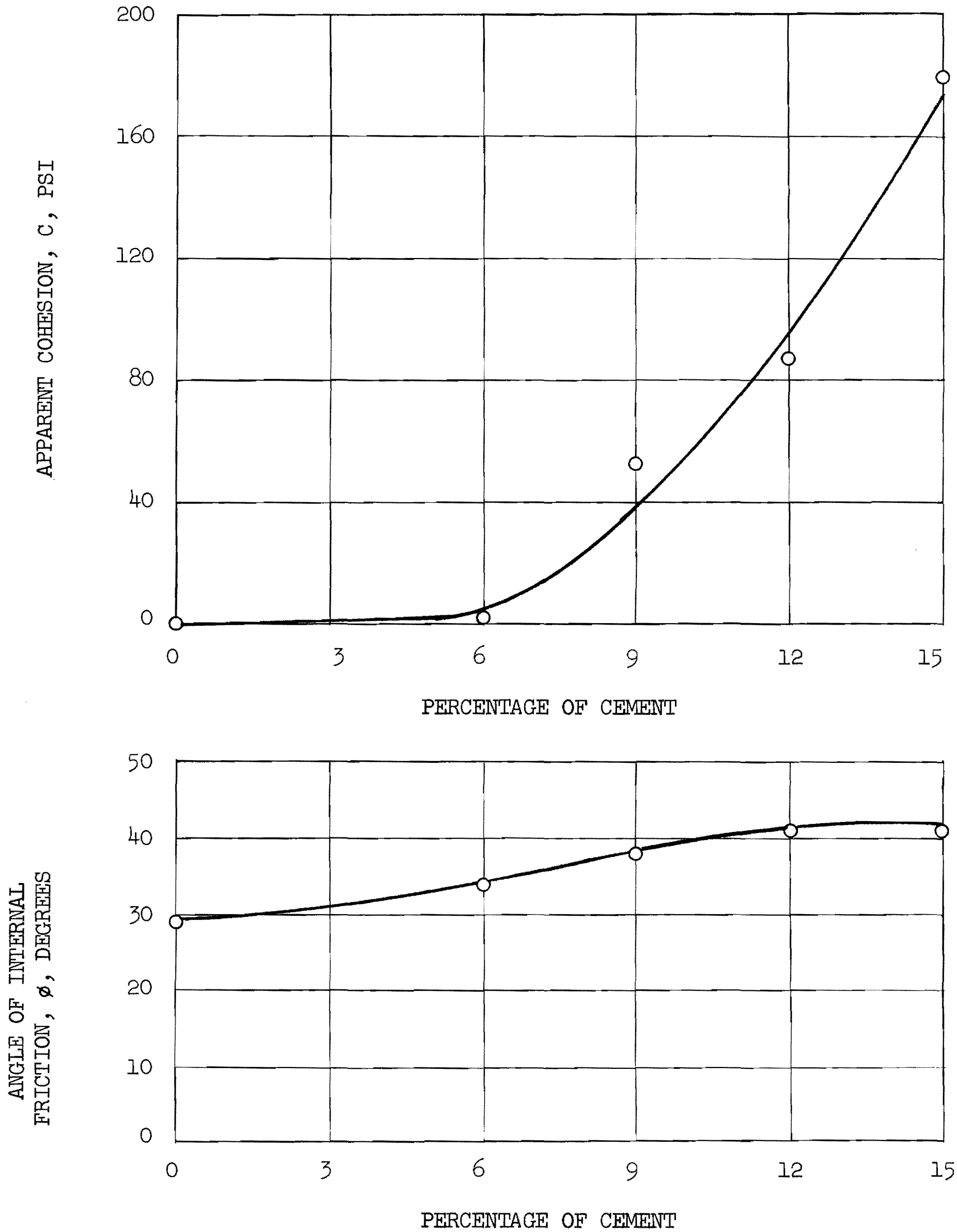


Figure 23. Apparent Cohesion and Angle of Internal Friction Versus Cement Content for Soil III.

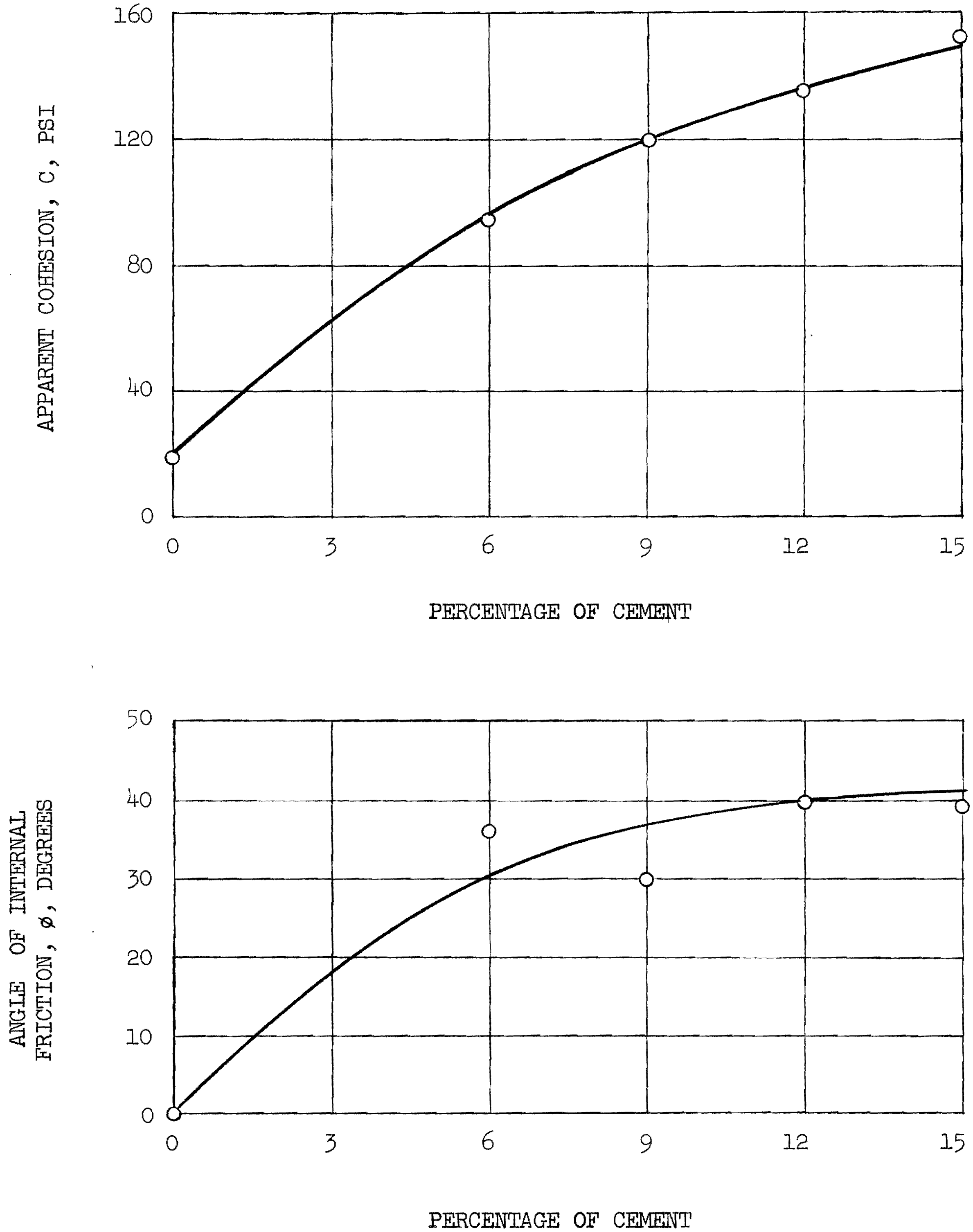
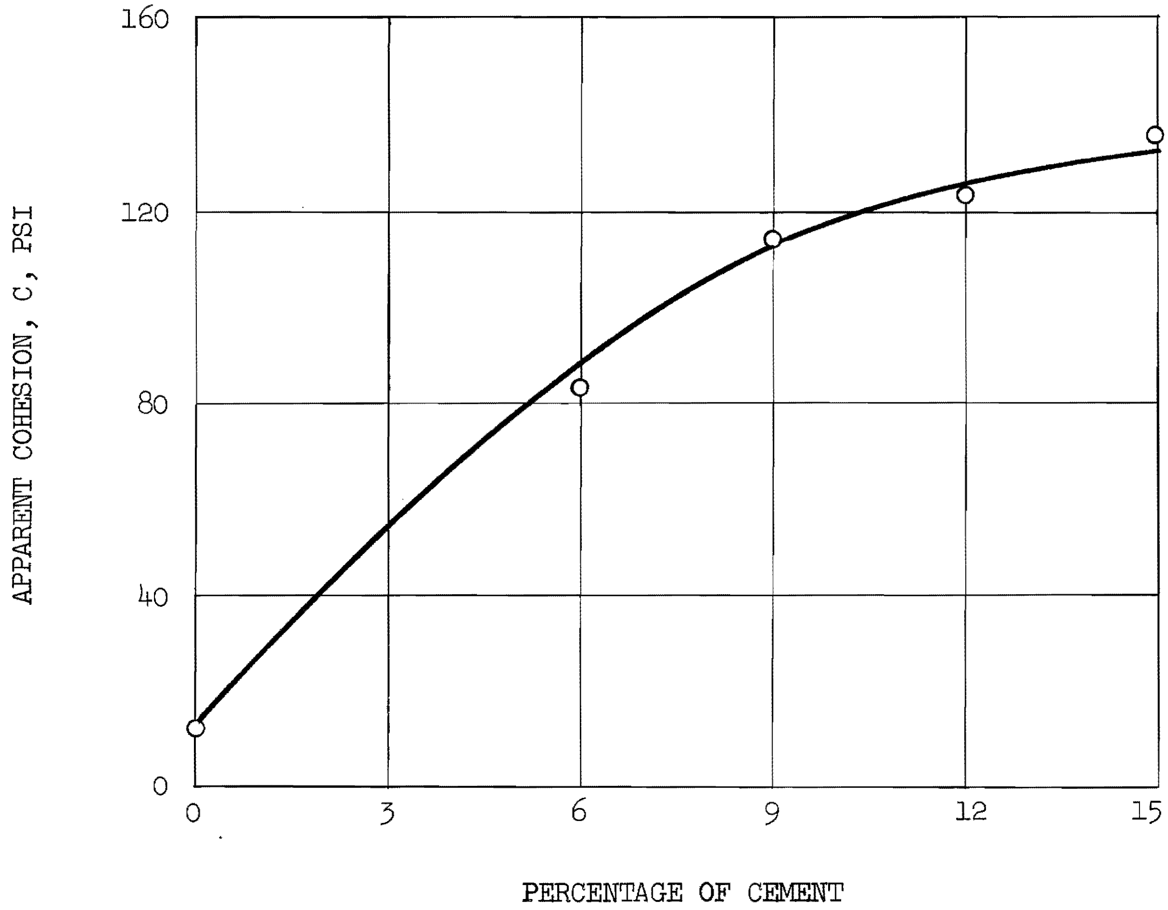


Figure 24. Apparent Cohesion and Angle of Internal Friction Versus Cement Content for Soil IV.



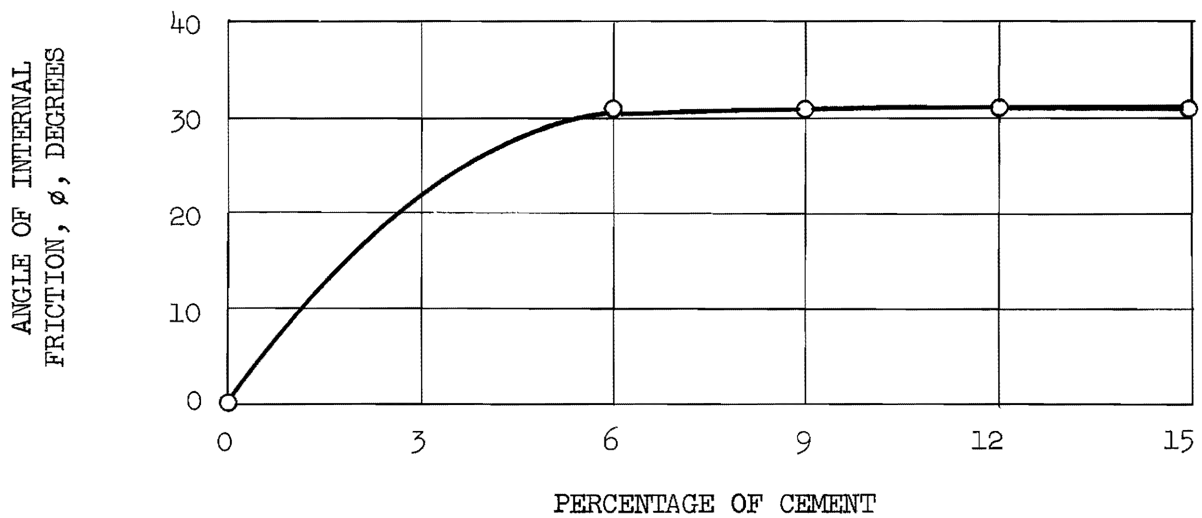


Figure 25. Apparent Cohesion and Angle of Internal Friction Versus Cement Content for Soil V.

cement caused an increase in angle of internal friction up to 41° at 12 per cent and remained the same for 15 per cent cement. There was little change in cohesion with 6 per cent cement but a rapid increase then up through 15 per cent cement where the cohesion was 180 psi.

Cohesion and angle of internal friction in Soil IV with no admixture were 19 psi and 0° respectively. The cohesion increased with increasing amounts of cement up to approximately 150 psi at 15 per cent. The angle of internal friction increased rapidly with approximately 30° at 6 per cent to a maximum of 40° at 12 and 15 per cent cement.

Soil V also had no internal friction with no admixture and a cohesion of 12 psi. The addition of cement caused an increase to 31° at 6 per cent and remained constant at that figure with increasing amounts of cement. Cohesion increased rapidly up through 9 per cent cement to 115 psi, then with a lesser rate of increase up to 136 psi at 15 per cent.

CHAPTER V

CONCLUSIONS

The following conclusions have been reached as a result of this study:

1. The addition of admixtures effects the maximum dry density and optimum moisture of a soil.
2. Strength of various soils can be improved by the addition of certain admixtures.
3. Portland cement effected the greatest increase in strength in all soils tested.
   a. Compressive strength increased with increased amounts of cement.
   b. The greatest rate of increase, in general, is at the higher cement contents.
   c. Increased strength varies directly with increased curing time.
4. The angle of internal friction, ø, and cohesion, c, is increased by the addition of portland cement.
5. The addition of 25% lime-flyash to a soil improved the strength of all soils.
   a. A 1:1 lime-flyash ratio gave the greatest strength improvement except with one soil where little change was noted from a 1:1 to 1:5 ratio.

b. Lime-flyash soil mixtures increased in strength with increased curing time.

6. Phosphoric acid caused a nominal increase in strength of all soils.

   a. The greatest improvement with this admixture was in the finer grain soils with the higher clay content.

   b. Two per cent acid gave a greater strength increase than did 1% acid.

   c. Strength after curing for 28 days was higher than after 7 days curing.

7. The addition of RC-3 caused negligible strength increases and in some cases caused a reduction in strength.

CHAPTER VI

RECOMMENDATIONS

The following recommendations are made for further study:

1. Further testing of the susceptibility of various soils to stabilization with portland cement.
2. An evaluation of the effects of variation in density and moisture on stabilized soils.
3. A study of the effects of exposure of high moisture conditions to stabilized soils during and after curing.
4. A study of volume change in soil-cement.
5. A study of cement stabilized soil-aggregate mixtures.
6. A determination of design requirements for various types of roads.

A P P E N D I X

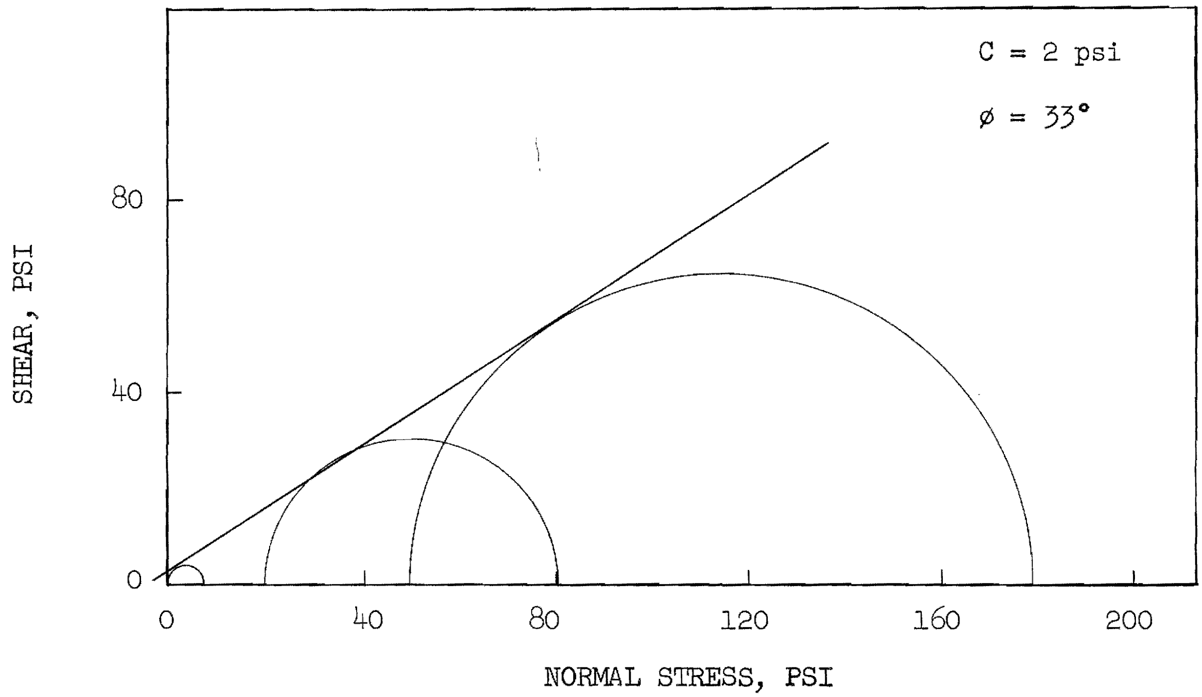


Figure 26. Mohr's Diagram for Soil I with no Admixture.

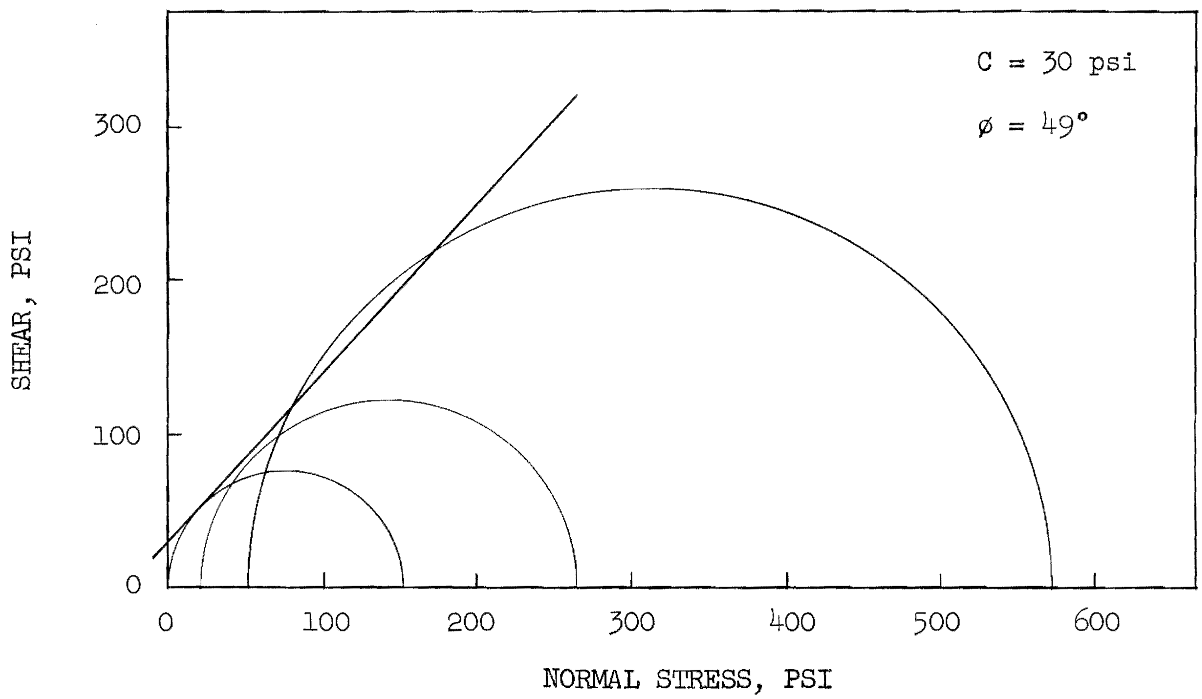


Figure 27. Mohr's Diagram for Soil I with 6% Portland Cement.

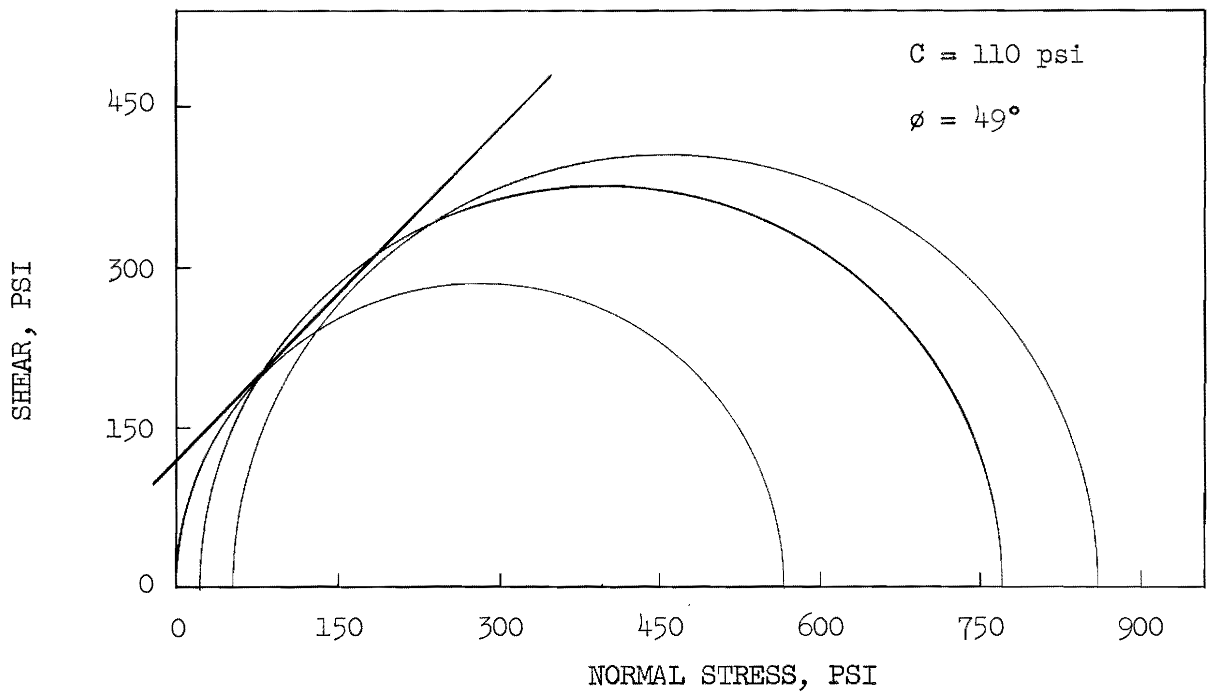


Figure 28. Mohr's Diagram for Soil I with 9% Portland Cement.

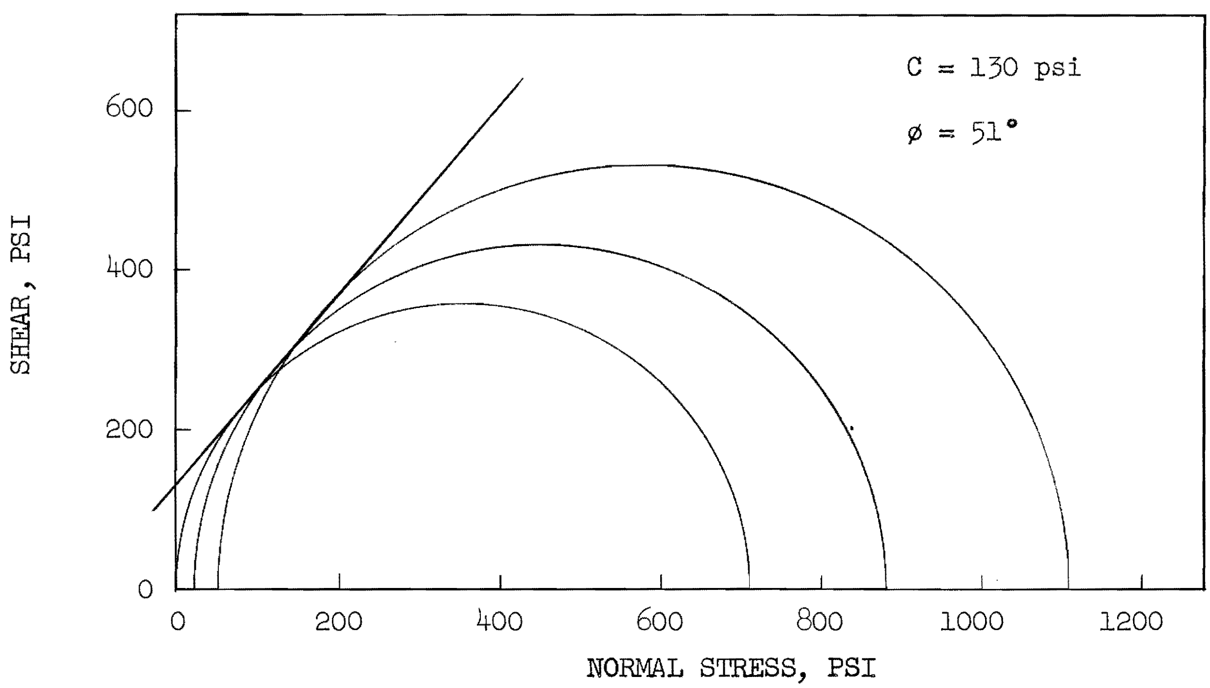


Figure 29. Mohr's Diagram for Soil I with 12% Portland Cement.

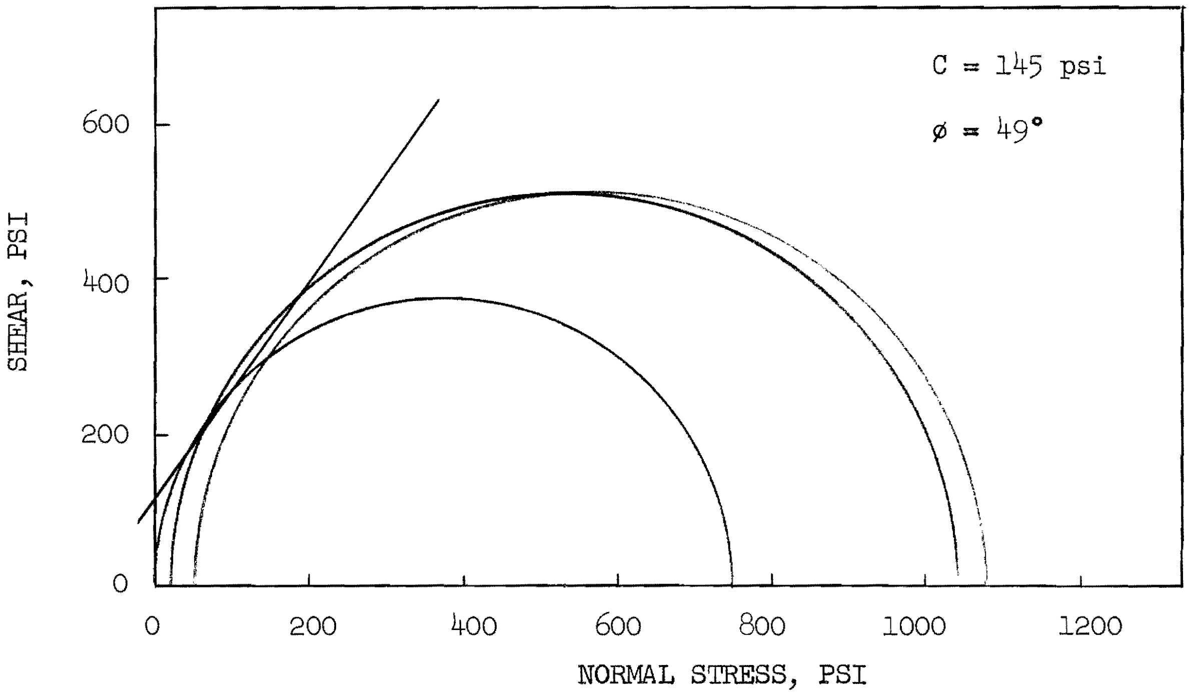


Figure 30. Mohr's Diagram for Soil I with 15% Portland Cement.

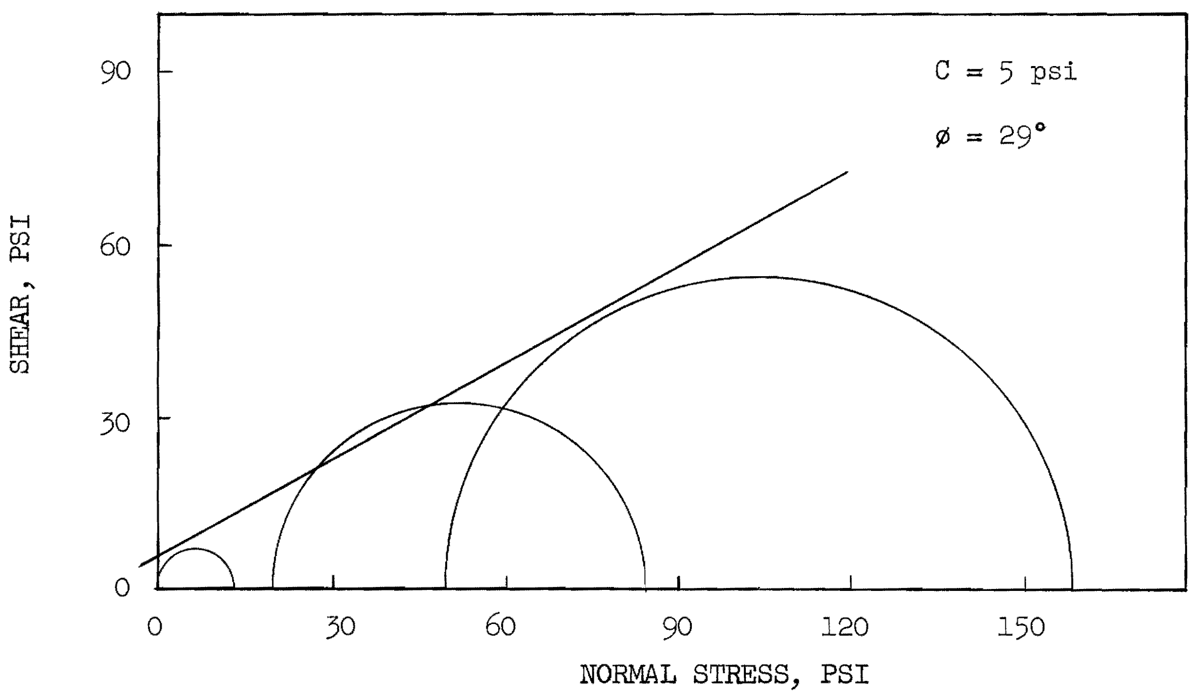


Figure 31. Mohr's Diagram for Soil II with no Admixture.

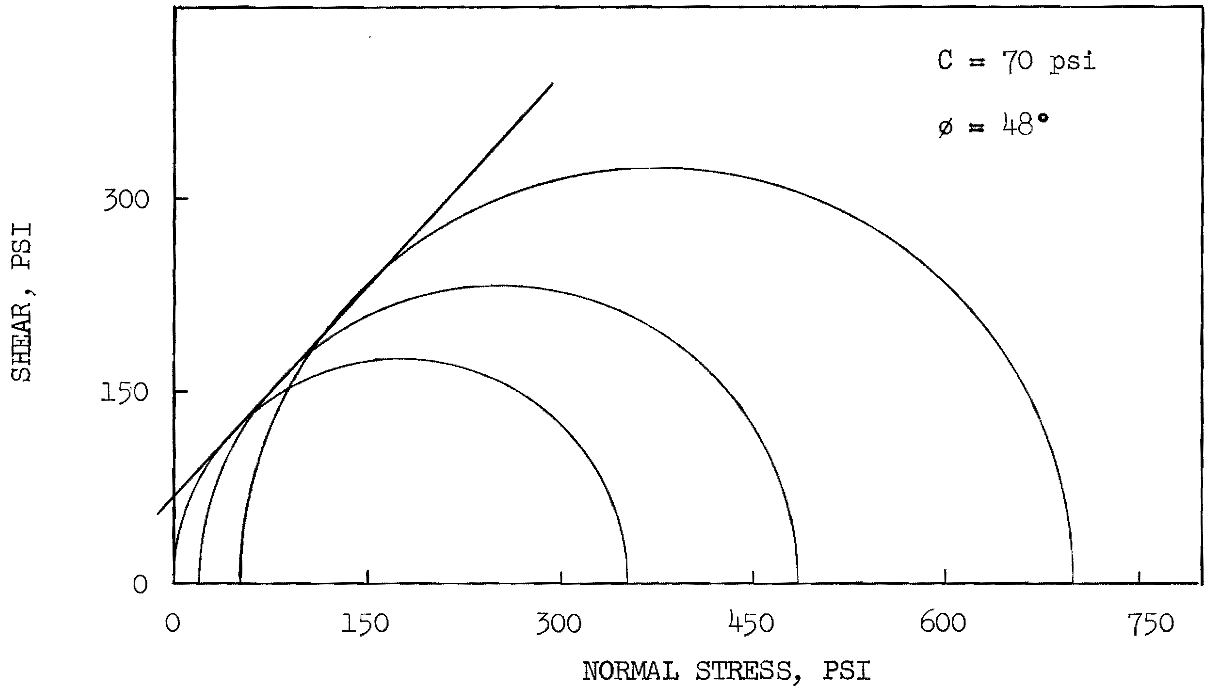


Figure 32. Mohr's Diagram for Soil II with 6% Portland Cement.

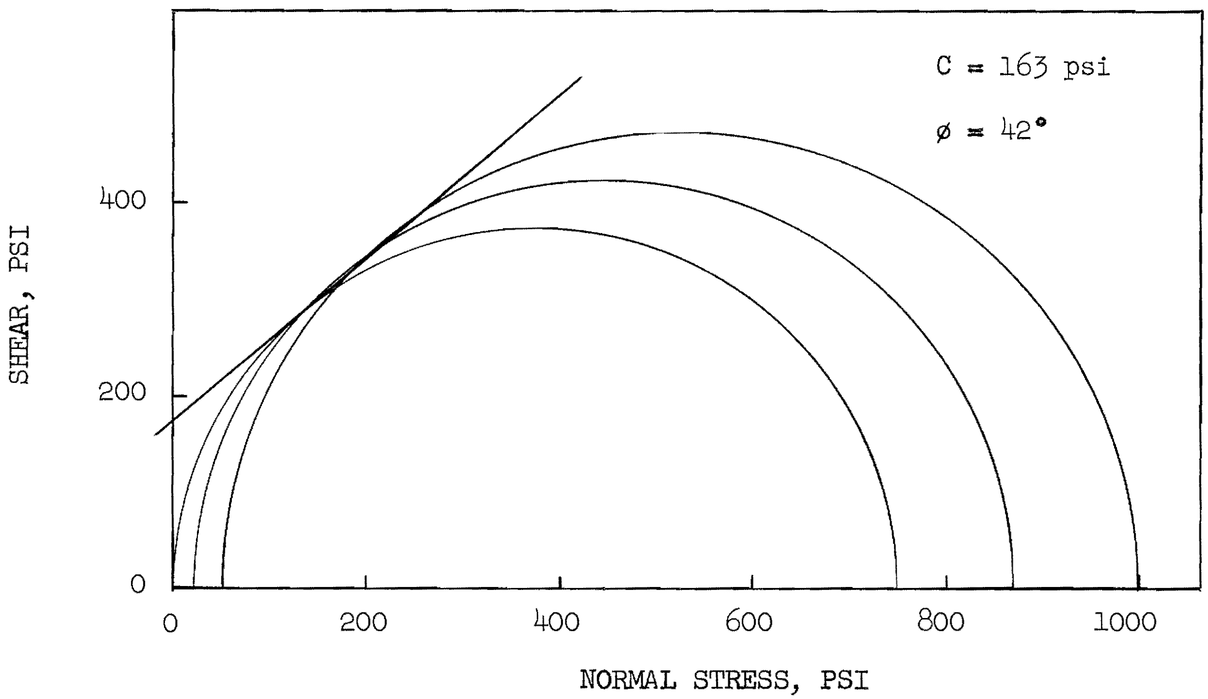


Figure 33. Mohr's Diagram for Soil II with 9% Portland Cement.

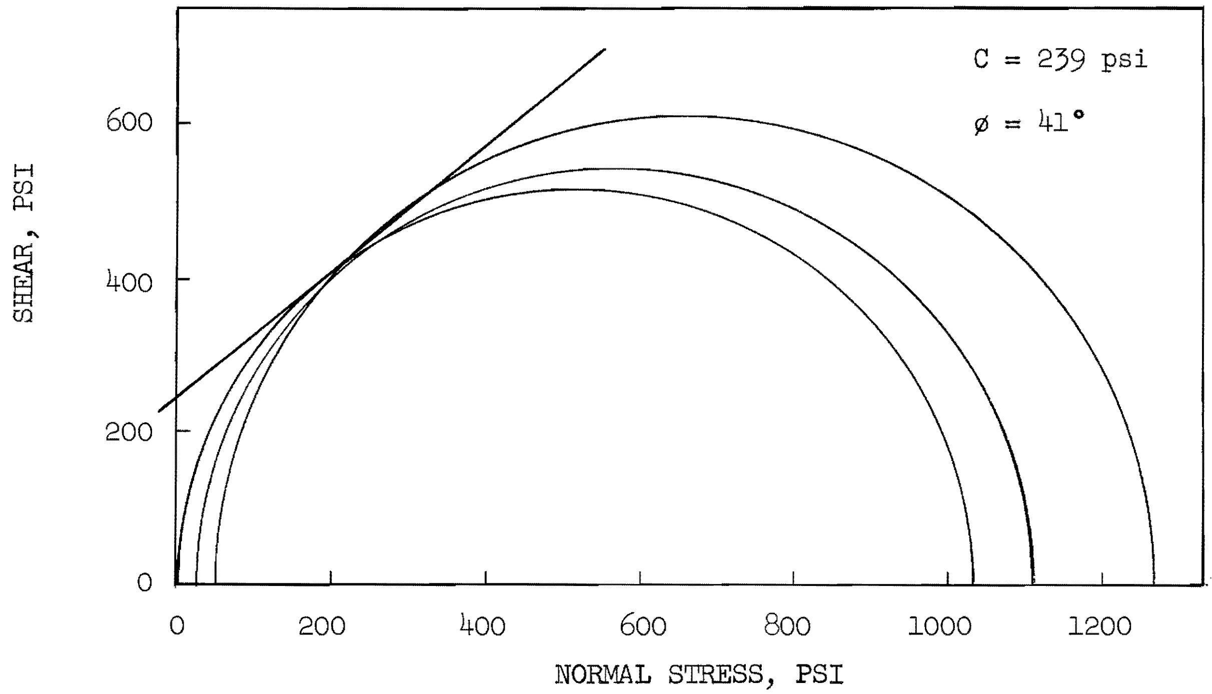


Figure 34. Mohr's Diagram for Soil II with 12% Portland Cement.

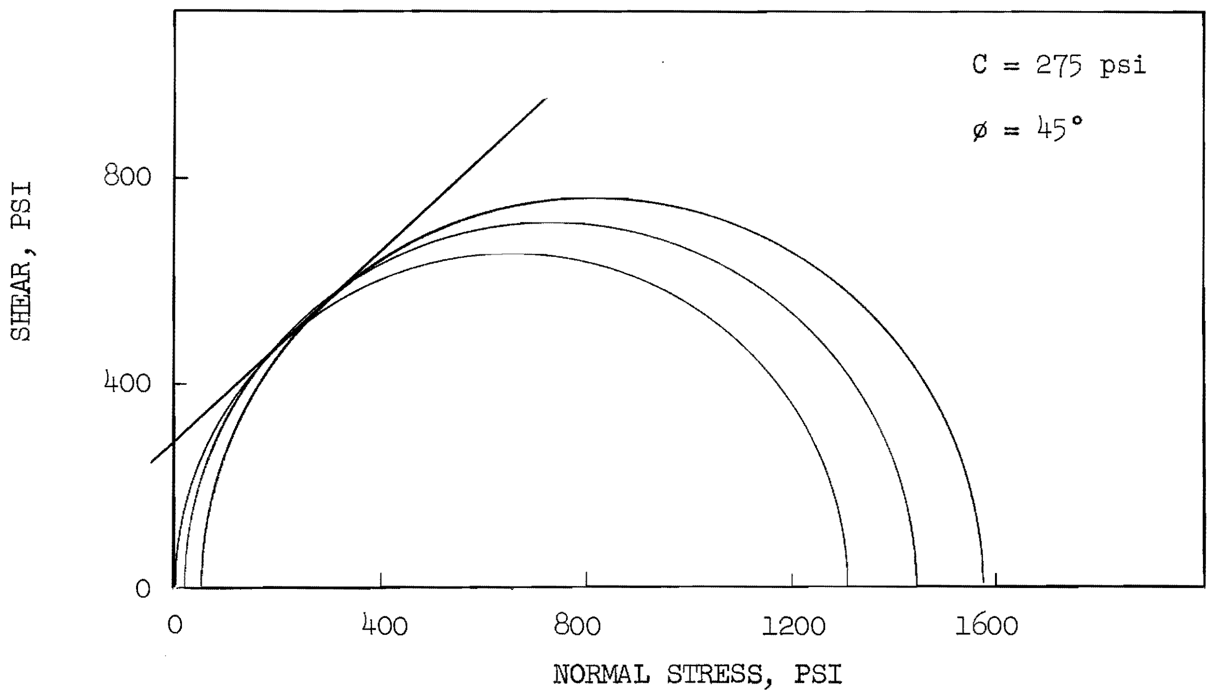


Figure 35. Mohr's Diagram for Soil II with 15% Portland Cement.

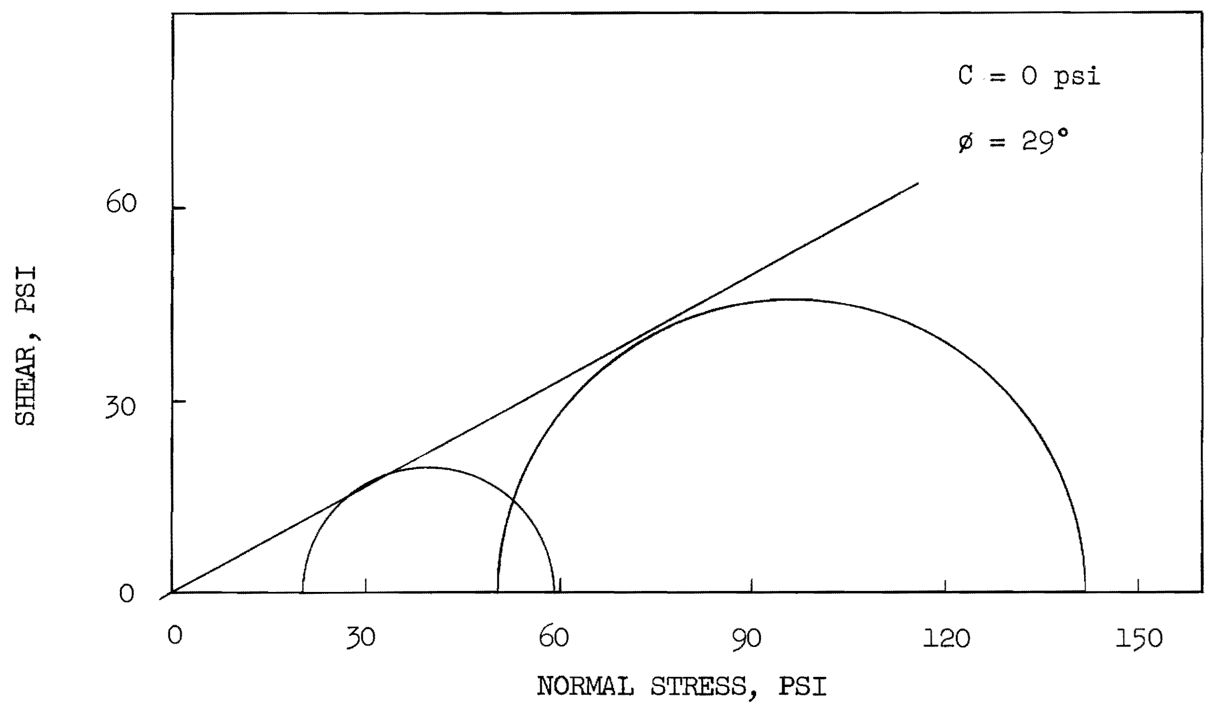


Figure 36. Mohr's Diagram for Soil III with no Admixture

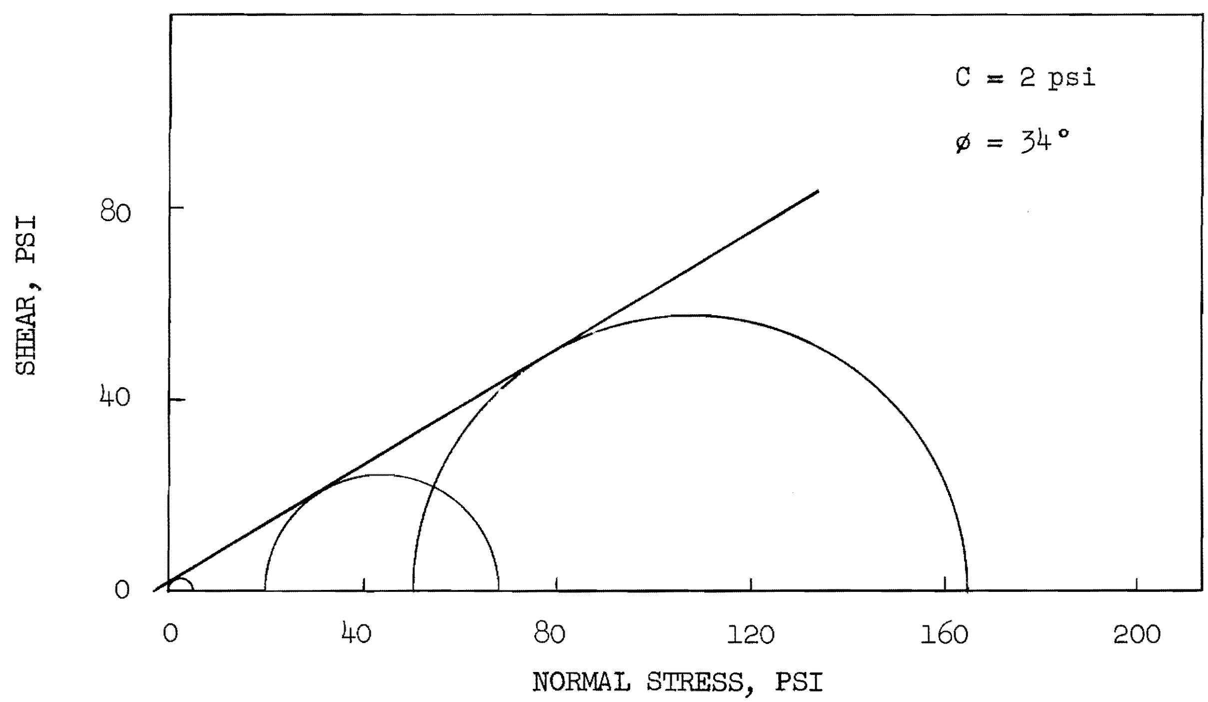


Figure 37. Mohr's Diagram for Soil III with 6% Portland Cement.

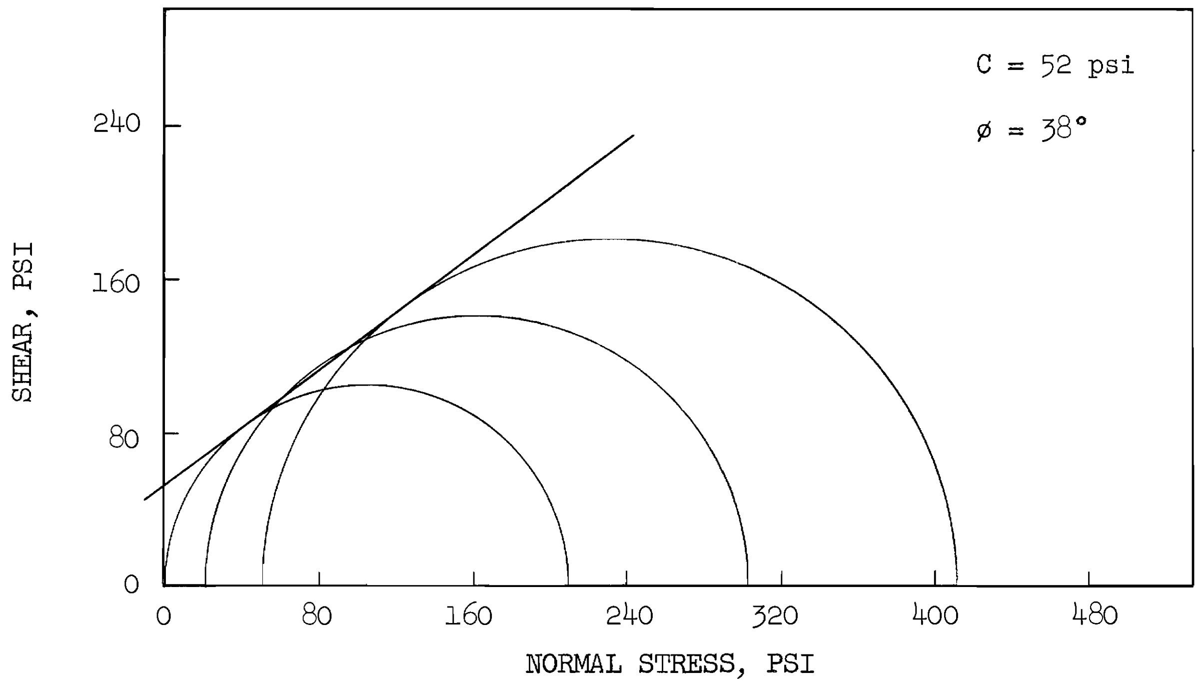


Figure 38. Mohr's Diagram for Soil III with 9% Portland Cement.

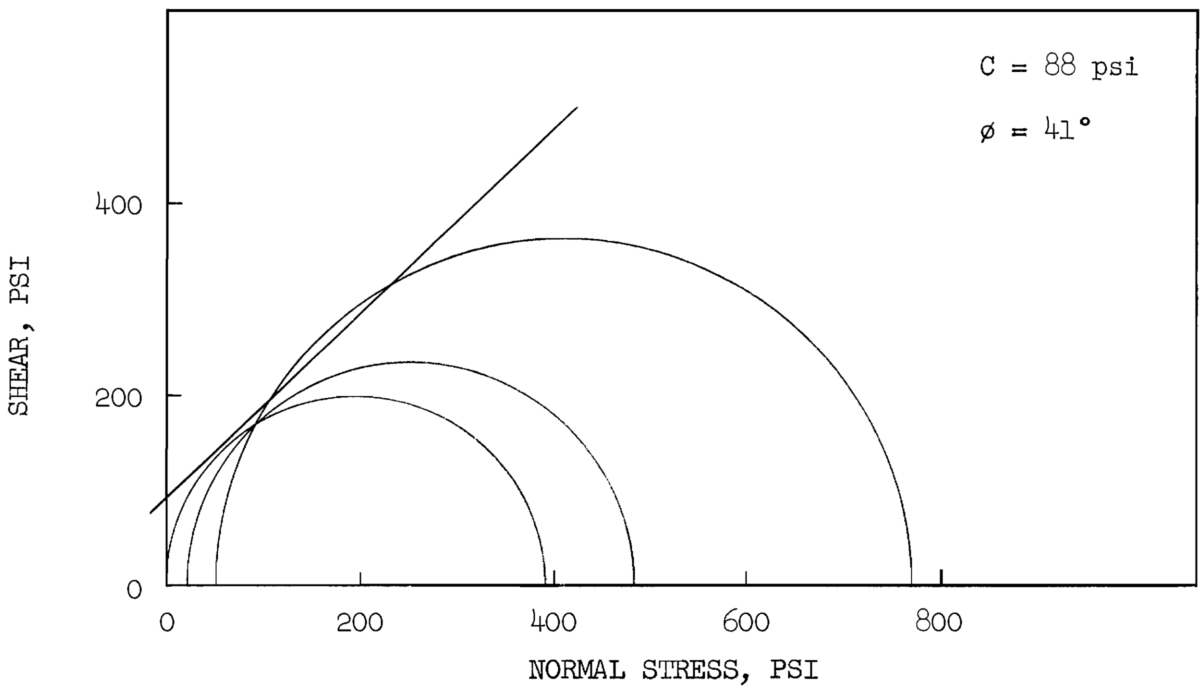


Figure 39. Mohr's Diagram for Soil III with 12% Portland Cement.

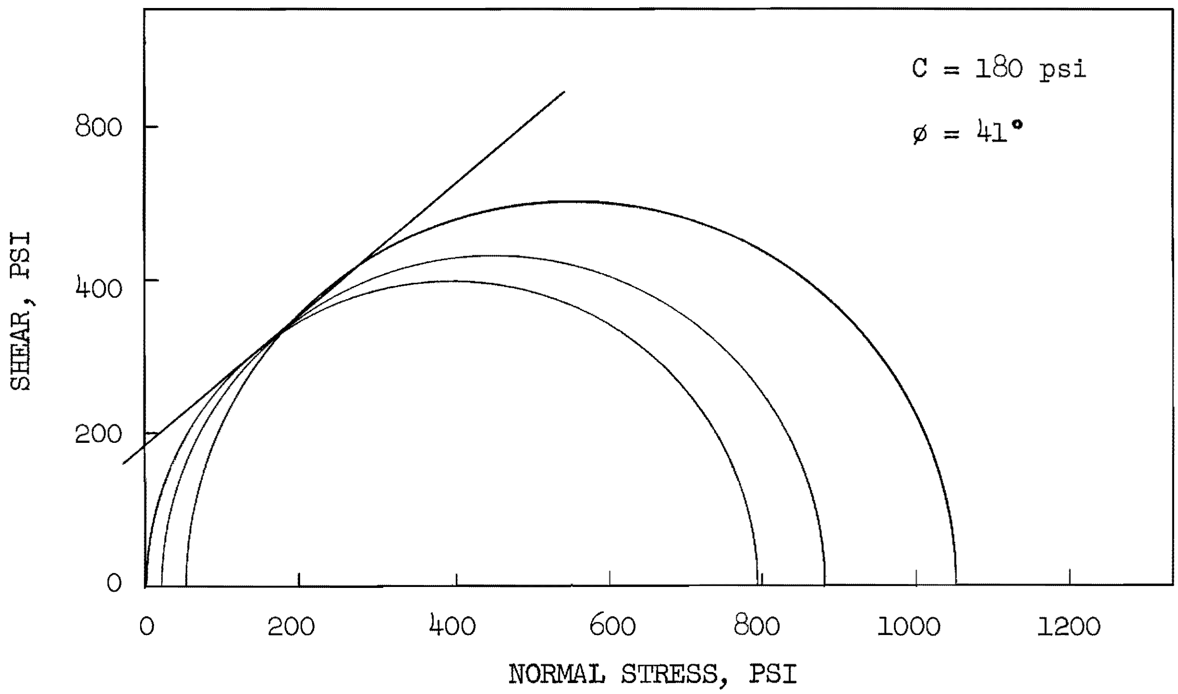


Figure 40. Mohr's Diagram for Soil III with 15% Portland Cement.

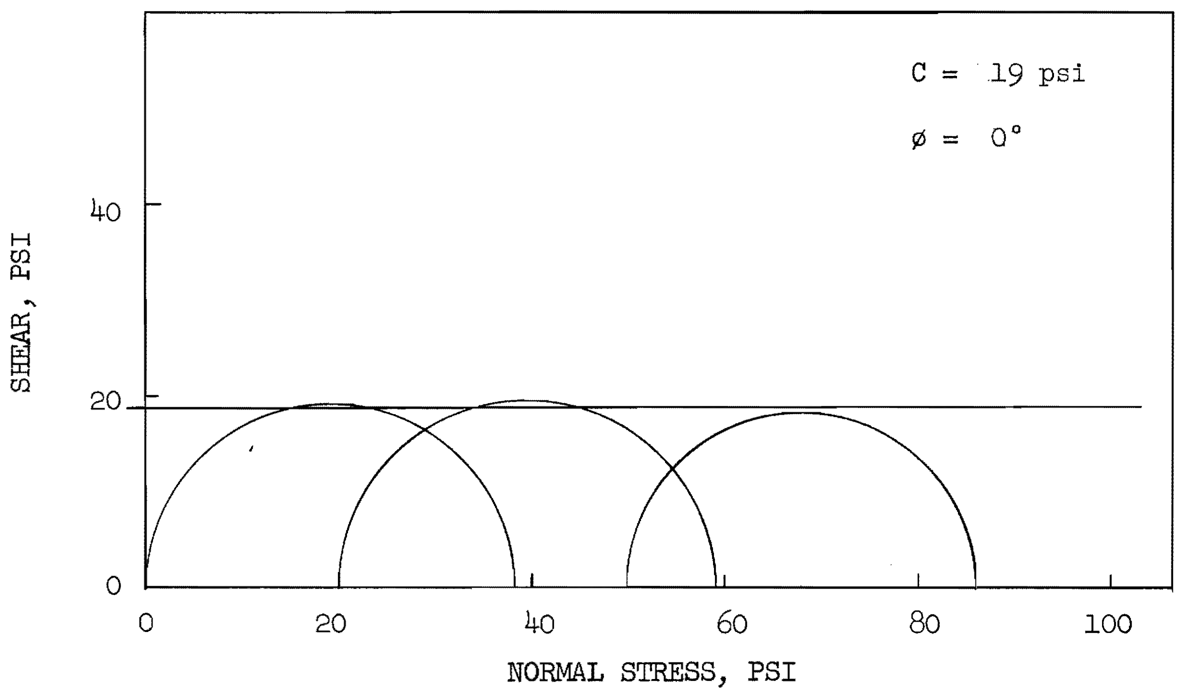


Figure 41. Mohr's Diagram for Soil IV with no Admixture.

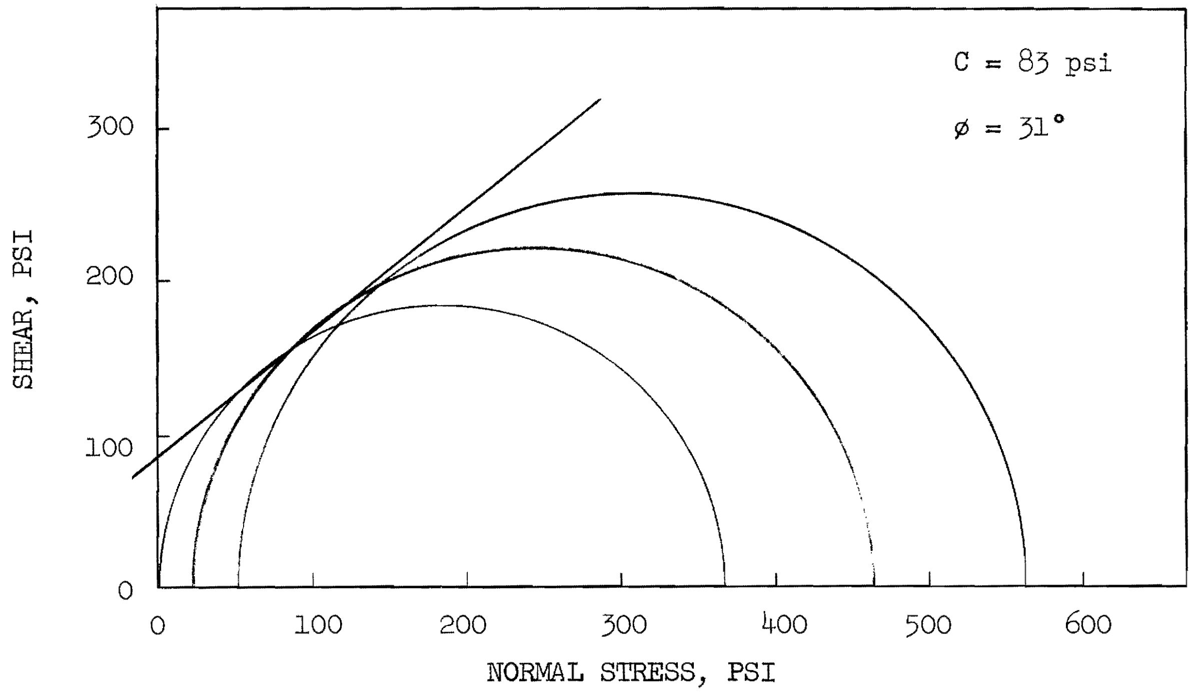


Figure 42. Mohr's Diagram for Soil IV with 6% Portland Cement.

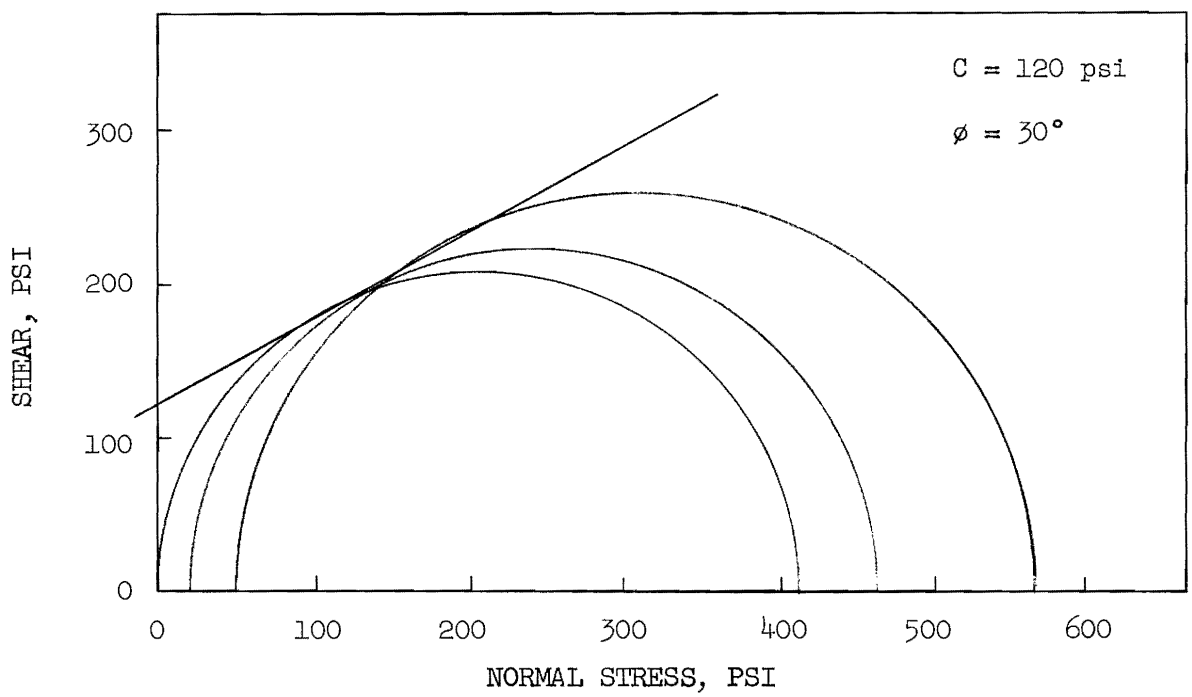


Figure 43. Mohr's Diagram for Soil IV with 9% Portland Cement.

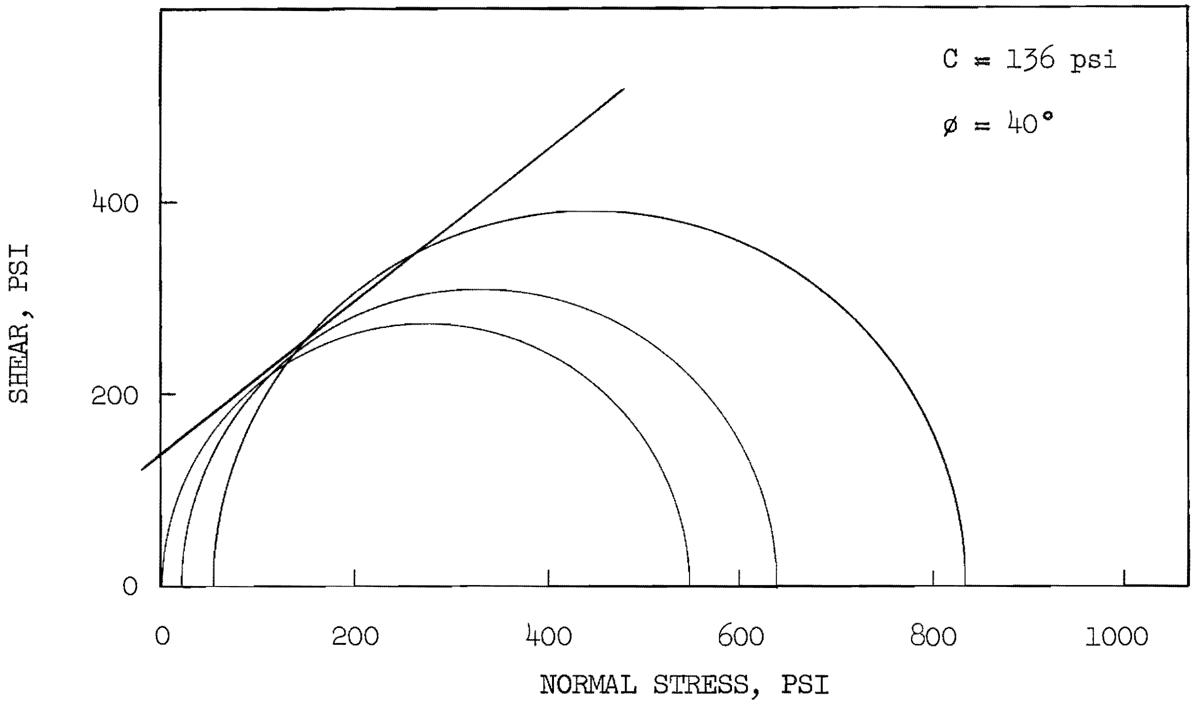


Figure 44. Mohr's Diagram for Soil IV with 12% Portland Cement.

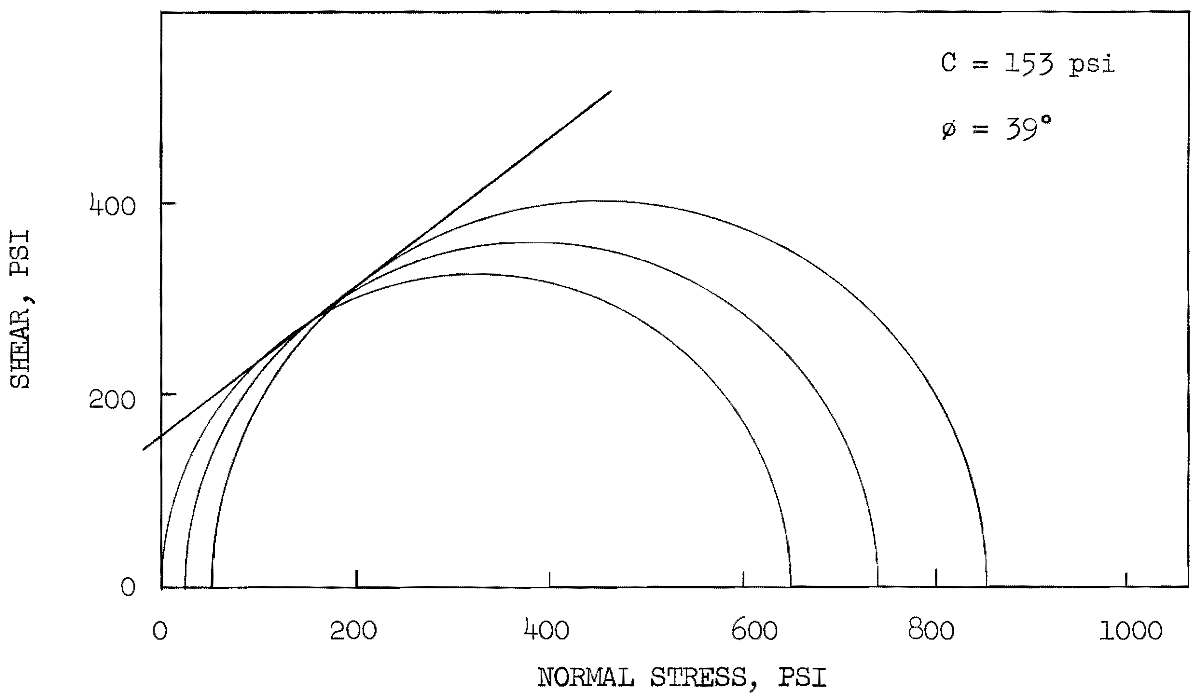


Figure 45. Mohr's Diagram for Soil IV with 15% Portland Cement.

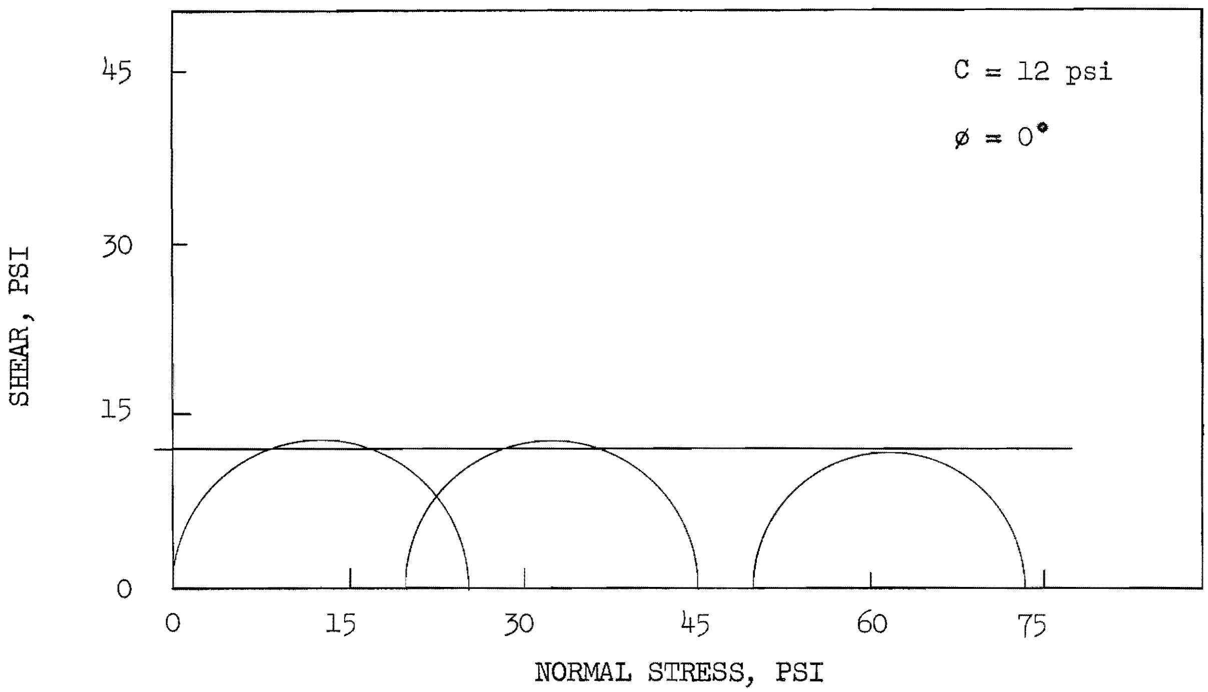


Figure 46. Mohr's Diagram for Soil V with no Admixture.

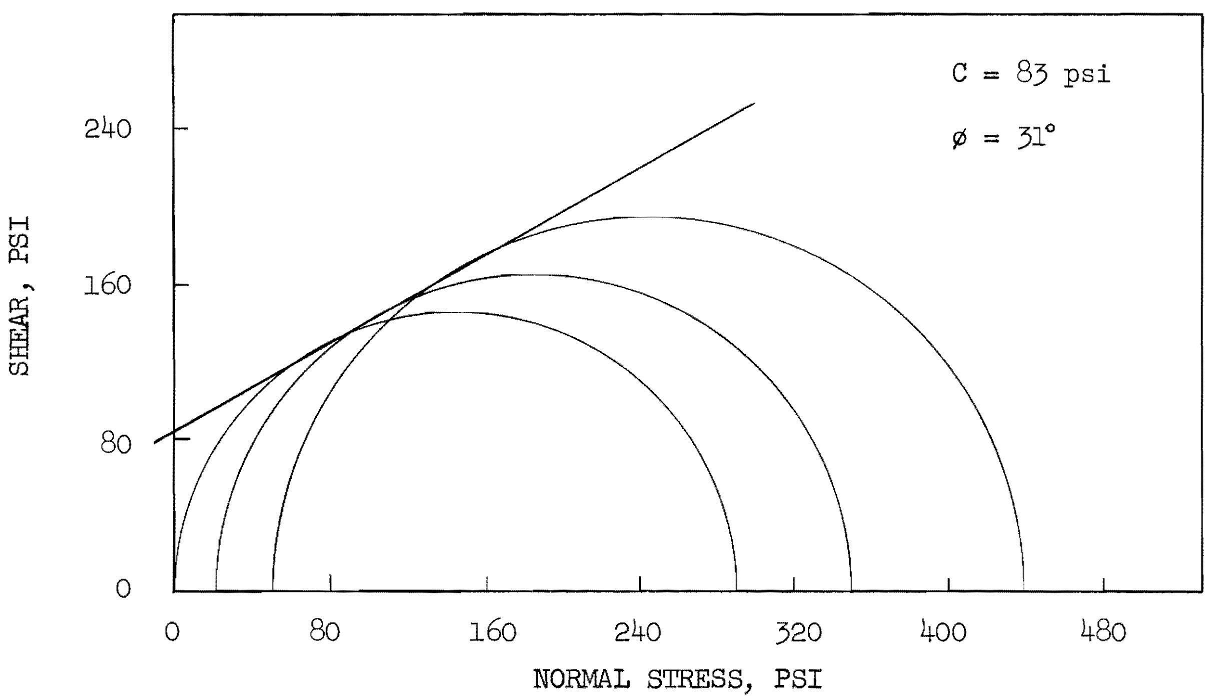


Figure 47. Mohr's Diagram for Soil V with 6% Portland Cement.

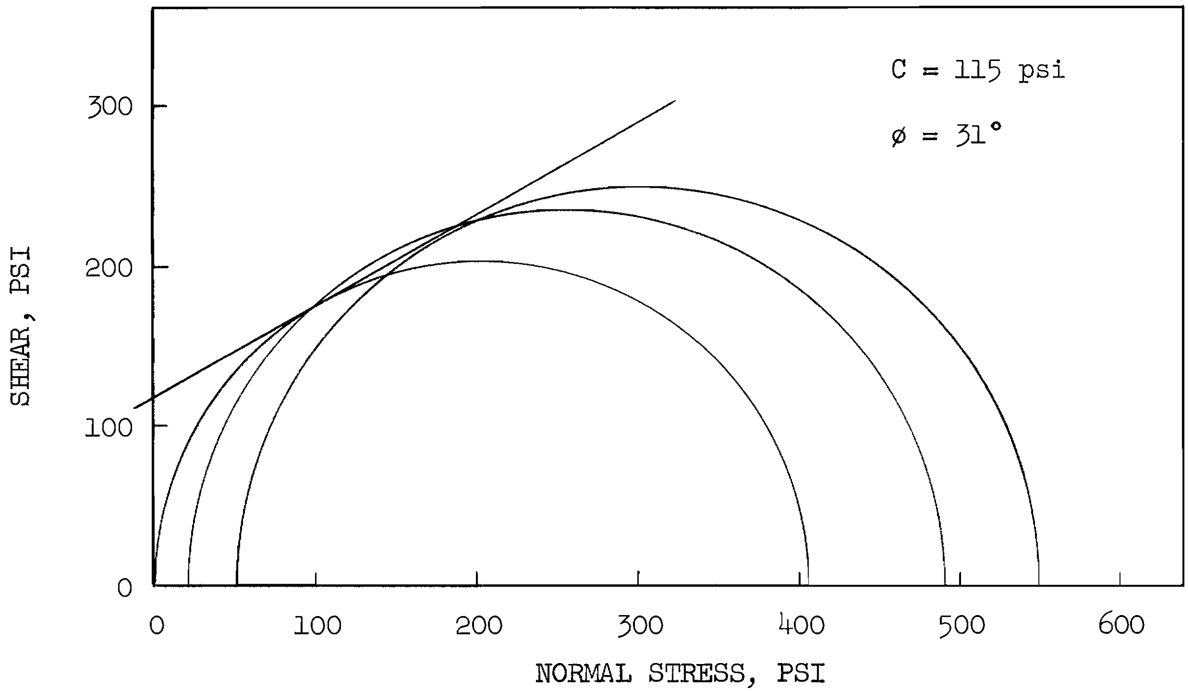


Figure 48. Mohr's Diagram for Soil V with 9% Portland Cement.

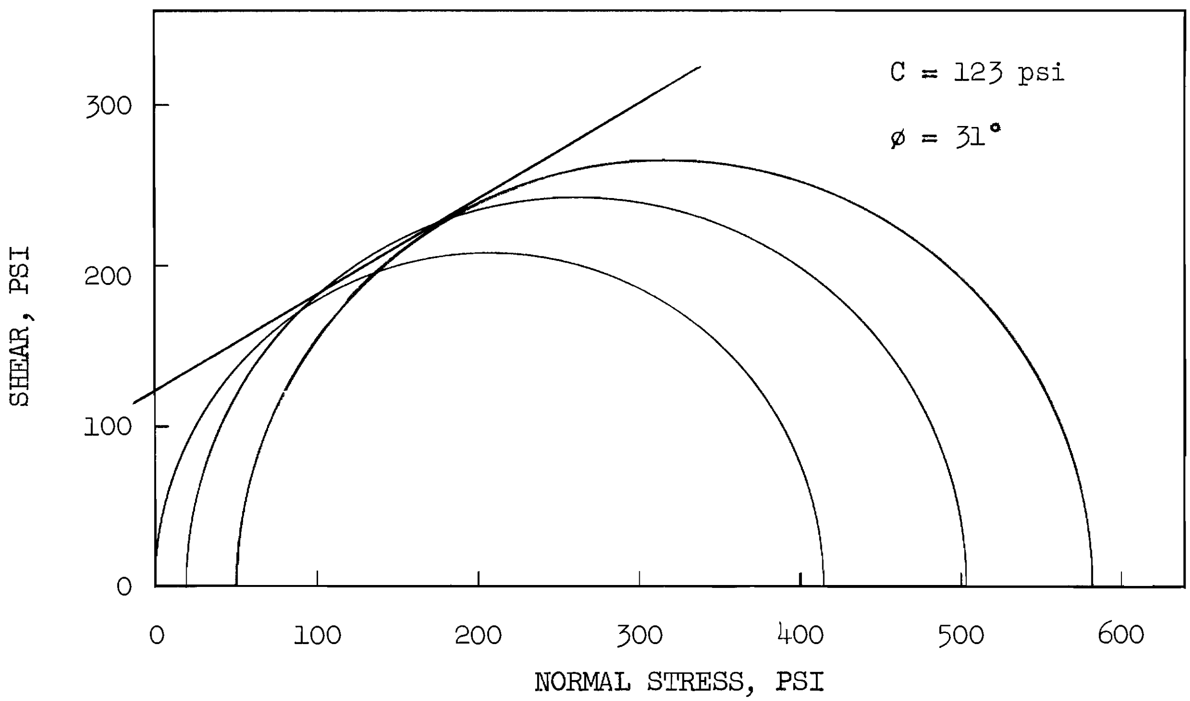


Figure 49. Mohr's Diagram for Soil V with 12% Portland Cement.

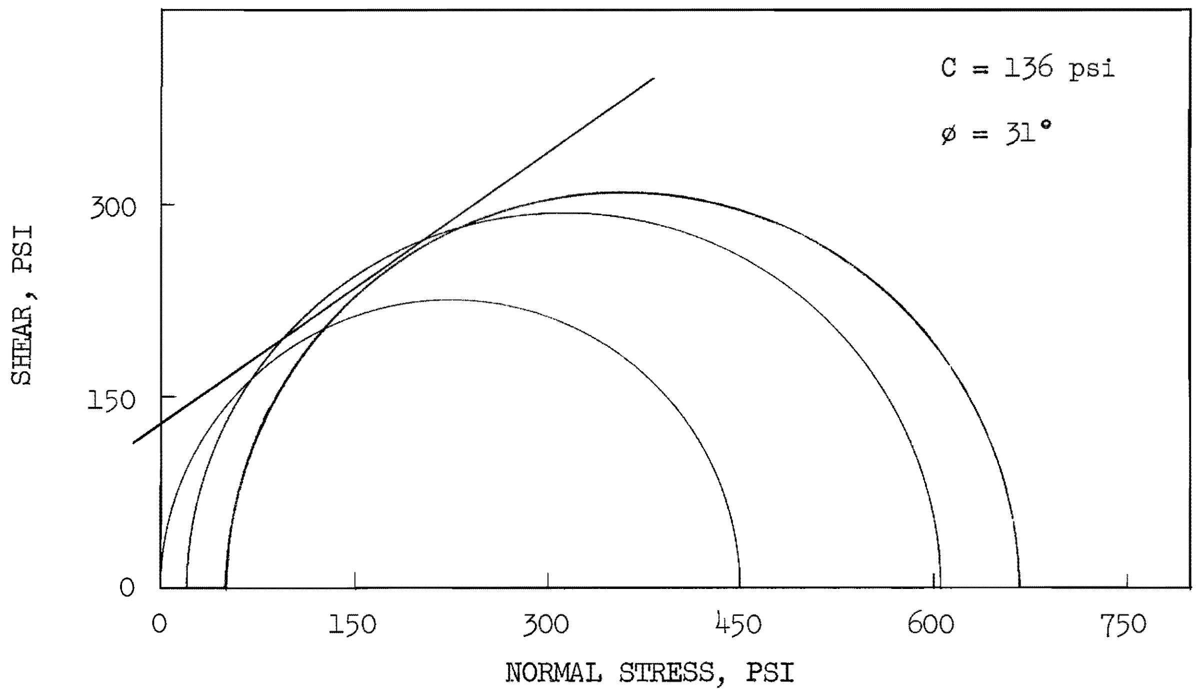


Figure 50. Mohr's Diagram for Soil V with 15% Portland Cement.

BIBLIOGRAPHY

1. Catton, Miles D., "Basic Principles of Soil-Cement Mixtures and Exploratory Laboratory Results," Proceedings, Highway Research Board, Vol. 17, 1937, pp. 7 - 31.

2. Winterkorn, Hans F., "A Fundamental Approach to the Stabilization of Cohesive Soils," Proceedings, Highway Research Board, Vol. 28, 1948, pp. 415 - 421.

3. Felt, Earl J., "Factors Influencing Physical Properties of Soil-Cement," Bulletin 108, Highway Research Board, 1955, pp. 128 - 137.

4. Goecker, W. L., Moh, Z. C., Davidson, D. T., and Chu, T. Y., "Stabilization of Fine and Coarse-Grained Soils with Lime-Flyash Admixtures," Bulletin 129, Highway Research Board, 1956, pp. 63 - 82.

5. Minnich, L. J. and Williams, Ralph, "Field Evaluation of Lime-Flyash-Soil Compositions for Roads," Bulletin 129, Highway Research Board, 1956, pp. 83 - 99.

6. "Stabilization of Soil with Asphalt," A.R.B.A. Committee on Soil-Asphalt Stabilization, American Road Builders Association, Technical Report, No. 200, 1953.

7. Benson, Jewell R., "Bituminous Soil Stabilization," Roads and Streets, Vol. 99, No. 5, May 1956.

8. Lyons, J. W. "Stabilizing a Problem Soil Cheaply," Engineering News Record, August 15, 1957, pp. 101 - 106.

9. Bishop, Alan W. and Henkel, D. J., The Measurement of Soil Properties in the Triaxial Test, London: Edward Arnold Publishers, Ltd., 1957.

10. Lambe, T. William, Soil Testing for Engineers, New York: John Wiley and Sons, Inc., 1951, pp. 99.

Technical Report

Project No. B-136 [HPS-1(54)]

AN INVESTIGATION TO DETERMINE THE ECONOMY AND PRACTICALITY OF USING VARIOUS TYPE SOILS TREATED WITH PORTLAND CEMENT OR OTHER ADMIXTURES FOR HIGHWAY CONSTRUCTION

By

Radnor J. Paquette and Charles Meyersohn

Contract with the State Highway Department of Georgia in Cooperation with the Bureau of Public Roads

January 1963

Engineering Experiment Station
GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia

ENGINEERING EXPERIMENT STATION
of the Georgia Institute of Technology
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CHAPTER I

INTRODUCTION

General.--The success of any highway pavement is primarily dependent on two factors: (1) the ability of the pavement system to withstand the most critical conditions of loading imposed on it, and (2) protection of the pavement components against the elements of nature to such a degree that the desirable properties of the structure are maintained throughout its design life.

The base and subgrade courses are the most critical components of the pavement system as they must, for economy reasons, be composed mostly of local soil. Since good, natural roadbuilding materials are not in abundance in many parts of the world, it is often necessary to improve the physical properties of the available material in order to fulfill the first requirement named above. For soils which have adequate strength under normal conditions but lose strength during periods of soaking, such as are caused by a high water table, it is necessary to use protective measures. The processes used to improve the strengths of natural soils or to preserve the natural strength properties of a soil come under the general heading of "stabilization".

Many different methods of stabilization have been used successfully in the past. Some of these methods are still in the development stages with economy being the biggest drawback to practical use. The most commonly used methods of soil stabilization today are: (1) mechanical stabilization, in which the gradation of the soil is altered by the blending in of other soil or crushed stone, thus producing a more compact and stable mixture; (2) cementing, in which portland cement is used to increase the cohesion; and (3) moisture resistance, in which bituminous material is mixed into the soil as a waterproofing agent in order to minimize swell and prevent loss of strength.[1] In Georgia, mechanical stabilization and cementing are the most widely used methods of soil stabilization.

Scope of the Project.--The three methods of soil stabilization described above have been used with and without success, the success often being due to a high factor of safety. Also, there is a tendency to apply designs and construction procedures which have been used elsewhere to local conditions. Because of the infinite variety of soils and climatic conditions existing, such generalizations should not be made in the field of soil stabilization.

This research project was undertaken (over five years ago) for the purpose of investigating the economy and practicality of certain materials as stabilizers for Georgia's highway bases and subgrades.

Summary of Previous Work Reported on the Project.--The first phase of this program[2] was initiated in order to evaluate four different Georgia soils stabilized with portland cement. This included a study of the susceptibility of these soils to cement treatment, measurement of the common physical properties of the various soil-cement combinations, and a determination of design requirements for soils which were to be used in future highway construction in Georgia. It was found that all four soils tested (referred to as Soils B, C, D, and E) were susceptible to cement treatment and that increases in compressive strength occurred in all soils, but to varying degrees. The effects of curing time on strength and of cement content on maximum density and optimum moisture varied with the particular soil.

The second portion of the research program[3] was concerned with comparing the effectiveness of various stabilizers on five different soils located in the

State of Georgia. The admixtures used were portland cement, RC-3 cutback asphalt, phosphoric acid (85% solution), and a lime-flyash combination. Three of the soils used, C, D, and E (now termed I, II, and III, respectively), were used in this work along with two new soils referred to as Soil IV and Soil V. The parameters used for evaluating the effects of the different stabilizing materials were confined and unconfined compressive strengths. A study of the influence of portland cement on cohesion and on the angle of internal friction was also made. The results of this study showed that portland cement affected the greatest increases in compressive strengths for all five soils tested. The addition of 25% lime-flyash resulted in strength gains for all five soils although these gains were much less pronounced than those obtained by the addition of portland cement. The lime-to-flyash ratio of 1:1 was the most effective in increasing compressive strength.

During the third phase of this research program[4] four more soils, designated as VI, VII, VIII, and IX, were subjected to the same type testing that had already been done for Soils I through V. In addition, the effects of molding moisture content on the 28-day compressive strength was evaluated for Soils I through VII. Also, a determination was made of the effect of various curing methods and of different moisture conditions, such as immersion or capillary soak, on the compressive strength of soil with or without cement added. The purpose of conducting the tests under these various conditions was to provide strength values which could be used to supplement existing Georgia Highway Department design criteria. Evaluation of the basic strength properties was desirable in order to develop a more rational thickness design of base courses using determinations of stress distribution from another research project being sponsored by the Georgia Highway Department. It was concluded from this study that all nine soils tested could be successfully stabilized with portland cement. The design compressive strength of the Georgia Highway Department, 300 pounds per square inch at 20 pounds per square inch confining pressure for a sample cured 7 days, was attained for each soil used; however, some soils required only 4 per cent while others required as much as 12 per cent cement. The strength tests performed on soil-cement specimens molded at equal densities but with varying moisture contents showed that, in general, the greatest compressive strength at 28 days is obtained either at optimum moisture or slightly higher. Two-day immersion proved to be the most damaging moisture condition for the majority of the soils as far as strength was concerned, although in a few cases capillary soaking was more detrimental to strength.

The fourth phase of this long-range soil stabilization project was concerned with the use of RC-3 cutback asphalt as a stabilizing agent.[5] Soils I through IX, with the exception of Soil V, were combined with various percentages of RC-3 and molded at maximum densities and optimum moisture contents and at densities less than maximum with corresponding moisture contents. The samples compacted at moisture contents less than the optimum were mixed at the optimum moisture content and dried back before compaction. The purpose of compacting samples at less than maximum density was so that the effect of density on compressive strength could be investigated. In was found from the work done that RC-3 cutback asphalt caused an increase in the density of the well-graded soils. On the other hand, the addition of RC-3 to a uniformly-graded soil resulted in a decrease in the density. An evaluation of the results of the compressive strength tests indicated that the maximum strength does not necessarily occur at the density and moisture content corresponding to the peak of the moisture-density curve. The RC-3 content found to be most beneficial to the compressive strength of the compacted soil was between 2 and 4 per cent.

The fifth phase of the research was composed of two distinct parts.[6] The first part was concerned with stabilizing soil with various combinations of stone screenings (from rock-crushing operations) and portland cement. Five different soils found in Georgia were utilized in this study. The percentages of stone screenings used were 0, 25, 50, and 75 and the percentages of portland cement added were 2, 4, 8, and 12. Moisture-density relationships were determined for each soil alone and for each soil combined with various combinations of stone screenings and portland cement. Use was made of the moisture-density relationships for molding samples for triaxial shear tests and unconfined compression tests. The data obtained from these tests were used to make up some typical design curves which can be utilized to determine the most economical combination of soil, screenings, and cement needed to produce a given compressive strength.

The second part of this phase was concerned with studying some of the problems involved in utilizing medium curing cutback asphalt, Grade 2, as a stabilizing agent. Because of the many variables involved,[7] a series of pilot studies were conducted before a regular testing program was organized. Some of the variables considered were asphalt temperature at the time of introduction to the soil, mixing time, and drying time before compaction. Also, during this period, an extensive testing program was systematized utilizing the results of the pilot studies.

Summary of Work Done from July 1, 1962 to January 1, 1963.--The object of the work done during this period was to determine the waterproofing properties of MC-2 cutback asphalt and the effect of this asphalt on strength.

As previously reported,[6] the moisture-density relationships were determined for Soil XI combined with percentages of MC-2 cutback asphalt ranging from 0 to 6. This was done for drying periods before compaction of 0, 3, and 6 days.

A testing program was designed and then initiated for determining the strength properties of various combinations of Soil XI, water, and MC-2 cutback asphalt; whole percentages of asphalt, from 0 to 6 were employed. The water content and dry densities used corresponded to the peak values on the moisture-density curves. Three variables were considered for each asphalt content used—drying time between mixing and compacting, curing time after compaction, and soaking time between curing and strength testing.

In order to achieve consistent results in the soaking of samples prior to strength testing, a multiple-sample soaking apparatus was designed and built. This device was designed specifically for the purpose of simulating capillary soaking occurring in compacted highway base courses. The amount of water absorbed and the vertical expansion of the samples were found.

Strength testing consisted of performing triaxial shear tests, using a confining pressure of 20 psi.

Following completion of the testing program, the results were tabulated and then analyzed. It was decided that some of the results were erratic; therefore, after a study of possible flaws in the testing procedure, a number of retests were made.

Upon completion of the work with MC-2, a similar study was begun using MC-4 cutback asphalt. Moisture-density relationships were determined for Soil XI and six different percentages of MC-4, compaction being performed immediately following the mixing process. Three different temperatures of MC-4 (100°, 125°, and 150° F) were utilized in this study.

Soil-Asphalt Stabilization.--Asphalts have been used successfully for stabilizing soils for many years. They function either as a cementing agent or a waterproofer. The cementing quality has been beneficial mostly in the stabilization of cohesionless soils. When used in moderately cohesive soils the asphalt tends to prevent the intrusion of water into clay-bound aggregations of soil, thus preserving the natural stability which the compacted soil has when in its best condition.

Soil-asphalt stabilization as discussed in this paper will concern cutback asphalts and emulsified asphalts; asphalt cements will not be discussed.

There is general agreement among those engaged in soil-asphalt stabilization research that the mechanism of this system can be explained in the two following ways:

(1) For cohesionless soils, the individual particles are coated with asphalt and the asphalt functions mainly as a binder, waterproofing being a secondary objective.

(2) For soils depending on hydraulic cohesion for strength, the capillaries are plugged with asphalt in order to protect this natural cohesion.

Waterproofing of soils with cutback asphalt is usually accomplished by blending and mixing asphalt with the wet soil; allowing some of the hydrocarbon volatiles to evaporate; compacting the soil; and, curing the compacted mixture so as to allow more evaporation of volatiles. Since the soil must be wet in order for the asphalt to distribute itself in a fairly uniform manner, the asphalt will not adhere to the soil particles initially due to the water films surrounding these particles. Instead, the asphalt will be in the form of small droplets (at least 0.01 mm. in diameter) distributed throughout the voids of the mix; any "excess" water also remains in the voids. During drying the water evaporates at a much faster rate than does the cutback material. If mixing is done periodically during the drying process the asphalt, if still fairly fluid, will distribute itself more uniformly throughout the voids. Upon compaction the asphalt films become thinner, plugging more capillaries. At this stage the water films surrounding the soil particles are still providing most of the cohesion. The curing period which generally follows provides further evaporation of water and hydrocarbon volatiles. If the water content becomes low enough some of the water films surrounding the soil particles may be replaced by asphalt films; this is not advantageous since the cohesion due to the asphalt is weaker than hydraulic cohesion. Once the compacted mixture is sealed, further evaporation will not occur. Theoretically, if each phase of the process described above was carried out during its optimum period, a successful soil-water-asphalt system would remain. Hydraulic cohesion would give the soil its natural strength while the asphalt would protect this strength by preventing the intrusion of water into the system.

Stabilization of soils with asphalt emulsions is generally accomplished by:

(1) blending and mixing of the emulsion with the wet soil which has a total water content near its optimum,

(2) compacting the mix, and

(3) curing.

In emulsified asphalt the particles of asphalt are as minute as the clay particles. They are carried in water as a suspending medium. In order to disperse the particles uniformly among the clay particles, it is necessary to separate the clay particles with water films. By adding the emulsified asphalt to this water, the asphalt particles are uniformly distributed. As the soil dries the water films decrease in thickness and the tension of the films increases. The asphalt particles are thus brought into contact with the soil particles under great pressure, and are spread in films of almost unimaginable thinness. Undoubtedly, a portion of the asphalt is absorbed by the clay particles under the great pressures. The stabilization of soil with emulsified asphalt requires that sufficient clay be present to act as a binding medium.

CHAPTER II

PAST RESEARCH IN SOIL-ASPHALT STABILIZATION

The first known experiments with asphalt as a soil stabilizer were done around the first part of the twentieth century. At that time, heavy petroleums were mixed with natural soils in an attempt to improve some of the basic physical characteristics of the natural material.

In 1929 C. P. Jensen, County Engineer at Fresno, California presented a paper on soil-asphalt stabilization at the Eighth Annual Asphalt Paving Conference. This report contained information as to the practice that had been used to construct the 3000 miles of soil-asphalt roads then existing in Fresno County. Some of these roads had been carrying a mixed traffic of as much as 1500 vehicles per day; truck loadings of 22,000 lb. on four wheels and 34,000 lb. on six or more wheels were allowed at that time.

Not long after soil-asphalt stabilization had been initiated in California, some of the midwestern states began using asphalt to stabilize some of their black soils. At the same time, similar projects were undertaken by several of the states along the eastern seacoast.

Many of these experimental projects proved to be successful in that the soils showed improvement over their untreated condition. However, the factors contributing to the success of these projects were not known. In other words, no design criteria had been established which would assure a successful soil-asphalt stabilization job, experience being heavily counted on in most cases. At the present time, there are still no design criteria available although many of the states have restricted the use of asphalt as a stabilizer for some of their soils, mainly the heavy clay soils.

As the higher type pavements began receiving more and more attention, interest in soil-asphalt stabilization faded somewhat. Many investigators throughout the country have worked on the soil-asphalt stabilization problem since that time. In most cases the conclusion has been that more research is needed.

Since the report by Jensen in 1929, many valuable papers have been published pertaining to soil-asphalt stabilization.

In 1935, C. L. McKesson presented a paper at the Highway Research Board meeting which concerned the stabilization of soil with emulsified asphalt.

The application of surface chemistry to soil-asphalt stabilization was first reported in 1934 by Hans F. Winterkorn. In 1936, Winterkorn reported on the surface-chemical aspects of the bond formation between bituminous materials and mineral surfaces.

A. M. Miller and E. W. Klinger issued a preliminary report in 1937 of studies in the use of bitumens in soil stabilization. This report was published in the January, 1937 Proceedings of the Association of Asphalt Paving Technologists.

A comprehensive study of the application of surface chemistry and physics to bituminous mixtures was reported by N. W. McLeod in 1937.

In 1939, J. C. Roediger and E. W. Klinger reported work done with cutback asphalts. The report was included in the Proceedings of the Association of Asphalt Paving Technologists.

E. B. Cape discussed the test methods used in the design and control of soil-bituminous mixtures in Texas in the Proceedings of the Association of Asphalt Paving Technologists, December 1940.

The 1940 Proceedings of the Association of Asphalt Paving Technologists contained a paper by Winterkorn and G. W. Eckert on the physico-chemical factors of importance in bituminous soil stabilization.

Also in 1940, Winterkorn presented to the Highway Research Board a paper entitled "Physico-chemical Testing of Soils and Application of the Results in Practice".

The Proceedings of the Association of Asphalt Paving Technologists, January 1942, contained an excellent paper by J. R. Benson and C. J. Becker. This paper, entitled "Exploratory Research in Bituminous Soil Stabilization", included the first detailed investigation of one of the most important features of soil-asphalt stabilization—the optimum condition of mixing of soil, water, and asphalt. Benson and Becker concluded that a definite soil system exists which changes its nature with the degree and method of mixing; and so far as waterproofing is concerned, the process passes through an optimum phase during the progression of mixing.

The work of Benson and Becker indicated that, as a rule, three general types of systems may be established in bituminous soil stabilization. These are:

1. An intimate soil-bitumen mixture in which each single soil particle is surrounded by a bituminous film and in which the shearing strength of the system is primarily governed by the consistency of the bituminous films; (bitumen in continuous phase).
2. A soil-bitumen mixture with bitumen as the continuous phase, the discontinuous phase consisting partly of single soil grains (such as sand or silt particles), partly of natural soil aggregations (these aggregations being of varying mechanical and slaking stability depending on the chemical nature of the cementing agents). The stability of

such systems is still governed mainly by the consistency of the bitumen, however, a greater degree of interlocking of the soil particles and aggregates is taking place with a possible greater contribution to shear resistance than in case 1.

3. A soil-water-bitumen system in which the soil particles are held together mainly by the water dipoles and by the interfacial tension (water-bitumen) on the surface of the water wedges; between the soil particles. Because the interfacial tension of water adjoining bitumen is smaller than that of water adjoining the shear resistance of such a system is smaller than that of an identical system without the bitumen; however, this shear resistance is considerably higher than that of either system 2 or 1.

The establishment of these three types of systems had been previously demonstrated by Winterkorn in his papers of 1934 and 1936 mentioned above.

The Highway Research Board Proceedings of 1942 contained a paper by V. A. Endersby which was entitled "Fundamental Research in Bituminous Soil Stabilization". Endersby, like Benson and Becker, found that control of the mixing phases is vital to the best results. In other words, control of field mixing methods should be maintained to make sure that optimum waterproofing is being obtained.

A. S. Michaels and Vytautas Puzinauskas investigated the effect of certain selected chemical additives on the stabilization of fine-grained soil with asphalt. This work was reported in Highway Research Bulletin 129.

Highway Research Bulletin 204 contained a paper by Moreland Herrin which was entitled "Drying Phase of Soil-Asphalt Construction". Herrin concluded from his work that soils stabilized with cutback asphalts should be dried out before compaction in order that high initial strength be obtained.

R. K. Katti, D. T. Davidson, and J. B. Sheeler presented a paper entitled "Water in Cutback Asphalt Stabilization of Soil" at the Highway Research Board meeting held in January 1959. The authors reached a very important conclusion as a result of their work. This was that "the percentage of mixing water required to produce maximum strength, maximum standard Proctor density, minimum moisture absorption during immersion, and minimum swelling is different for each property mentioned".

CHAPTER III

THEORY OF SOIL-ASPHALT STABILIZATION

In comparing the histories of the soil-stabilizing admixtures most used today—cement, lime, salt, calcium chloride, asphaltic materials—it is found that stabilizing with asphaltic materials is one of the older methods. Yet, there is less known about the theory of asphalt-soil stabilization than any of the other methods.

The reason that so little is known about the theory of asphalt-soil stabilization is quite simple—this method involves many more variables than any other method. In stabilizing with cement, lime, etc., the chemical reactions that take place, and the physical changes that occur, generally can be explained. This is not true, however, of the process of asphalt-soil stabilization.

Let us consider the variables involved in stabilizing soil with cutback asphalt. In the first place, the asphalt itself is produced in 15 different standard types; i.e., five different grades in each of the rapid-curing, medium-curing, and slow-curing asphalts. Each of these 15 liquid asphalts has certain characteristics not possessed by any of the others. The rate of evaporation of hydrocarbon volatiles is one of these characteristics; it is usually measured by the Standard ASTM Distillation Test. Viscosity is a property that varies within any one type of liquid asphalt, each of the three types of cutback asphalts having five different viscosity ranges. It is interesting to note here that temperature effects these two basic characteristics—temperature of the cutback asphalt during mixing and temperature of the air after mixing and after compaction. This is of great importance because the state of the cutback asphalt in the soil at the time that the compacted mix is sealed will have a great influence on whether or not the soil-asphalt system is successful in its intended function, whether it be waterproofing or cementing.

Even if it were known which type and grade of cutback asphalt would be best to stabilize a certain soil with, the problem of determining the amount of asphalt to add would still have to be solved. So far, no method has been devised for forecasting this figure, the most suitable amount being determined by trial and error. However, because economics are involved here, the range of asphalt contents can be narrowed down to feasible limits without a great deal of work.

Another important variable is the soil itself. What influence do gradation, clay content, etc. have on the susceptibility of a particular soil to being stabilized with asphalt? Some investigators have found that certain soils can best be stabilized with certain types and grades of asphalt. However, a soil having apparently similar properties but located in another area may best be stabilized with some other asphalt material. Generally, there is no theory to explain this.

The moisture content of the soil at the time that the asphalt is added is another variable. Benson and Becker classified the moisture contents of the soil that could be used for adding asphalt as follows:

1. The minimum moisture mixture where the moisture is usually limited to hygroscopic moisture. This moisture may not exceed 2 per cent in sandy materials, but may be considerably higher in heavier soils. This mix classification is most commonly associated with cutbacks or fluxed asphalts.

2. The wet or slurry type of mix, in which the soil, particularly the heavier type, is reduced to semi-fluid consistency with water in order that the asphaltic material may be intimately mixed with the soil particles. This type of mix may be used with either the cutback asphalts or emulsions.

3. The third type of mixture is neither wet nor dry. The moisture content of the soil for this type of mix is such that a certain intermediate degree of cohesiveness exists in the soil mass, yet the soil may be "fluffed" by agitation, producing a condition allowing the addition and mixing of asphaltic material in the form of cutbacks.

Many different moisture contents are specified by the various agencies involved in soil-asphalt stabilization work. Four of these are:

1. Optimum moisture for maximum density of the soil itself.
2. Moisture content at the "fluff point" which is actually a small range of moisture contents at which the soil is considered to contain the greatest amount of air voids, as judged visually.
3. Optimum moisture for maximum density of the soil minus cutback asphalt content.
4. One-half optimum moisture for maximum density of the soil.

These moisture contents were found by experience to be, under certain conditions and with certain soils and asphalts, most beneficial in promoting a homogeneous mix of soil and asphalt. Again, there are no theoretical bases involved in choosing these values of moisture content.

Closely related to the moisture content variable is the degree and method of mixing of the soil-water-asphalt blend. It was found by Benson and Becker that so far as waterproofing is concerned, the soil-water-asphalt system passes through an optimum phase during the progression of mixing. Some of the phenomena connected with this phase-mixing process were described by Endersby as follows:

1. The oil is distributed through the soil in large masses having little waterproofing value.
2. These masses are broken down until a substantial "plugging" action is evident.
3. The oil is distributed over aggregations or "cells" of soil particles.
4. At first these cells are too large; and the entrance of water into individual cells imperfectly waterproofed breaks down the soil as a whole.
5. An optimum condition is reached where the cells are small enough so that failure of individual scattered members is not serious.
6. The cells become too small, hence imperfectly waterproofed, because the oil is distributed too thinly.
7. A condition approaching "intimate mixing" is reached. In this region some stabilizations continue to improve; others break down completely.

Benson and Becker also found that there are vital differences in the phase aspect of different mixers. However, these investigators did find that satisfactory correlations could be made between laboratory mixers and field-mixing methods.

Another variable to consider is the degree of evaporation of water and hydrocarbon volatiles which should be allowed before and after the mix is compacted. Many agencies doing soil-asphalt stabilization leave this decision to the engineer in charge. Others specify a certain period of drying before compaction and curing after compaction. Herrin, recognizing the importance of the drying and curing of the mixture, made an investigation to determine the rate of evaporation of the water and hydrocarbon volatiles from a soil-asphalt mixture and the resulting effects on the stability and other basic properties of the mixture. Herrin considered the following four variables:

1. The amount of initial mixing water.
2. The amount of initial cutback asphalt.
3. The amount of drying time in the loose state before compaction.
4. The time of curing after compaction.

In this investigation, the mixtures were dried in an oven and the compacted specimens were cured in an oven. Whether the methods of drying and curing have any appreciable effect on the basic properties of a soil-water-asphalt mix is open to question. The author of this investigation drew two important conclusions from his work. One was that "soils stabilized with cutbacks need to be dried out before compaction, not to adjust the liquid content for optimum compaction but to provide high initial stability. After compaction, additional curing results in even more stability".

CHAPTER IV

DESCRIPTION OF MATERIALS AND LABORATORY PROCEDURES

Materials.--Soil XI was used in all the testing described in this report. This soil was obtained from Crisp County, Georgia and is presently being used in the subbase construction of Interstate 75, the top four inches being stabilized with cutback asphalt.

Soil XI is a non-plastic, grayish-brown, clayey, silty sand. The sand portion of the material is fairly well graded. According to the soil classification method of the AASHO (American Association of State Highway Officials), it would be classified as A-2-4(0).

The medium-curing cutback asphalts used, Grade 2 and Grade 4, were obtained from the Shell Oil Company refinery in Savannah, Georgia. Distillation, penetration, viscosity, and specific gravity tests were conducted periodically on these materials. It was found that these properties varied very little for the asphalts used throughout the research work. Typical values of the properties determined are shown in Table 1. All asphalt used met the specifications of the American Society for Testing Materials for medium-curing cutback asphalt Grades 2 and 4.

Physical Testing of Soil.--All soil brought to the laboratory for testing was stored in barrels. Soil to be used the following day in testing was passed through a No. 4 sieve, placed in a large pan, and allowed to air-dry. Clay lumps were broken down to pass the No. 4 sieve and thoroughly mixed in with the soil while roots were discarded.

The first testing performed on the soil was the grain size analysis. This test was conducted as specified in AASHO Designation T 88-57. The results

TABLE 1

ANALYSIS OF MEDIUM-CURING CUTBACK ASPHALT

Characteristics	ASTM Test Method	Grade 2	Grade 4
Flash Point (Open Tag), °F	D 1310	203	221
Furol Viscosity at 140° F, seconds	D 88	163	
Furol Viscosity at 180° F, seconds	D 88		194
Distillation:	D 402		
Distillate (% of total distillate to 680° F)			
To 437° F		0	0
To 500° F		30	12
To 600° F		77	63
Residue from distillation to 680° F, Volume % by difference		72	81
Penetration on residue at 77° F, 100 gm., 5 sec. (ASTM Method D 5)		239	313

obtained are as follows:

U.S. Standard Sieve No.	% of Total Passing
10	99
40	71
60	47
100	28
200	16

The hydrometer analysis showed the soil to contain 11% silt and 8% clay.

The specific gravity of the soil was determined according to AASHO Designation T 100-60 and was found to be 2.59.

The moisture-density relationship was obtained following the procedure described in AASHO Designation T 180-57. The maximum dry density was found to be 122.4 pounds per cubic foot at a moisture content of 9.4 per cent.

For determining the liquid and plastic limits of the soil, AASHO Designations T 89-60 and T 90-56 were used, respectively. The soil was determined to be non-plastic.

Proportioning of Soil and Cutback Asphalt.--Cutback asphalt was added to Soil XI as a percentage of dry soil. For example, a mix containing "2 per cent MC-2" would be composed of 10 pounds of dry soil and 0.2 pound of MC-2. The percentages used were 1, 2, 3, 4, 5 and 6.

Moisture-Density Relationships for Soil-Asphalt Mixes.--It has been found from past experience that the drying of a soil-asphalt mix, before compaction, has a significant effect on the density and strength properties of the compacted mix. This effect is related to the evaporation of hydrocarbon volatiles in the cutback asphalt and the evaporation of water. The influence of the individual losses is not known.

Since it is generally believed that the asphalt volatile loss causes an increase in the strength of a compacted soil-asphalt base course, soil-asphalt mixes are usually allowed to "dry back" before being compacted. The length of this drying period varies; for example, some states specify a minimum drying period while other states leave this to the judgment of the engineer in charge. It should be mentioned here that the effects of air temperature and humidity on the degree of volatile loss is usually neglected in determining the length of the drying period to be used in the field. Also, it is interesting to note that researchers in the past have ignored these two important variables.

Because of the appreciable effect of drying the mix, moisture-density relationships were determined for mixes of Soil XI and MC-2 dried back for 3 days and 6 days before compaction as well as for mixes compacted immediately following mixing. The latter drying period, although perhaps excessive for practical purposes, was obtained so that a usable upper limit could be established. Other investigators[11] have used short periods of time for oven-drying mixtures before compaction. It is believed that good correlation between oven-drying and air-drying is difficult, if not impossible, to establish. For this reason, all drying of mixes was done at air temperatures in the approximate range of 65-85° F.

The procedure used for preparing and compacting a moisture-density sample was as follows:

1. The hygroscopic moisture of the soil was estimated by the use of a "Speedy" Moisture Tester manufactured by the Alpha Lux Co., Inc. In this method, a 6-gram sample of soil is placed in the tester body with a specified amount of calcium carbide. The gas pressure created by the chemical reaction of the reagent and water in the soil

activates a gage which directly gives the percentage of water in the soil sample. The amount of water to be added to attain the desired water content was then found.

2. The desired amount of soil was weighed to the nearest 0.01 pound and the desired volume of water to be added was measured to the nearest 10 milliliters. The soil was placed in a 10-quart capacity mixing bowl and, as the water was being slowly added, mixed mechanically 45 seconds by a Hobart C-100 mixer with a flat blade. The mixing was stopped to scrape the sides of the mixing bowl and the blade and then mixing was continued for another 45 seconds. A moisture sample of about 100 grams was taken from the bowl and weighed.
3. The cutback asphalt, which had been kept in a water bath at the constant desired temperature, was removed from the bath and the desired amount of asphalt added by weight to the soil-water mix; the proper amount being weighed to the nearest 0.01 pound. The mixing process described above was now repeated.
4. The moisture sample was dried in an oven at 230° F for 24 hours and the actual water content determined. If the actual water content was more than ± 0.3% different from the desired water content, a new mix was made up and the above procedure repeated.
5. The mixes that were to be dried were placed in tin buckets having a diameter of about 4 inches (see Figure 1); thus, the surface area exposed to air was the same as the compacted sample. The mixes of soil and MC-2 were allowed to dry, no attempt being made to control the humidity or the temperature of the air.

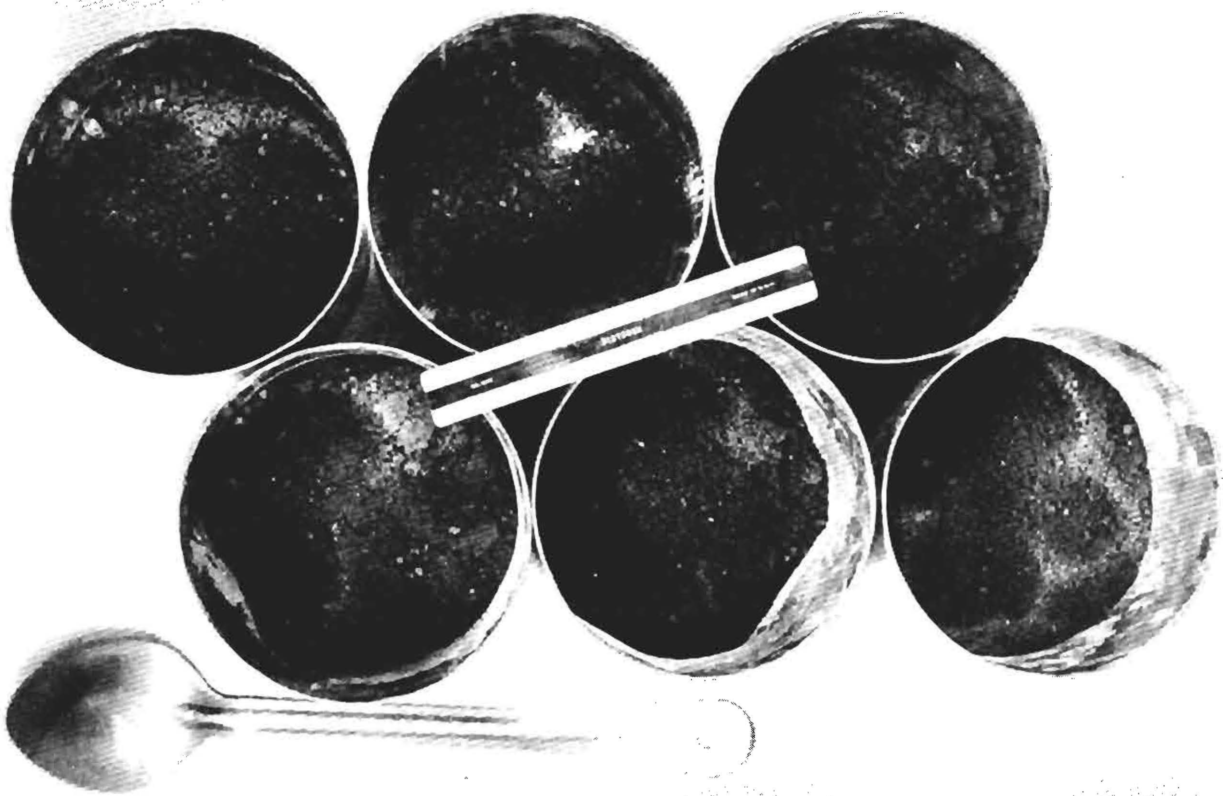

Figure 1. Drying of Soil-Asphalt Mixes before Compaction.

6. After the mix had been dried for the desired amount of time, it was compacted. The method outlined in AASHO Designation T 180-57 was followed.

7. The dry density was determined by extracting the asphalt from the compacted mix and weighing the remaining dry soil. The procedure used was similar to that described in ASTM Designation D 1097-58.

At least five different water contents were used for each combination of soil and asphalt. After a mix had been compacted, it was not used again.

The total mixing period of three minutes described above was established as a result of a pilot study previously conducted.[6] This same mixing procedure was used for the moisture-density testing of Soil XI with MC-4. Because of time limitations, the only moisture-density relationships established for Soil XI and MC-4 were those involving no drying of the mixes. Three different temperatures of MC-4 were used—100°, 125°, and 150° F.

<u>Determination of Compressive Strength and Water Absorption of Compacted Specimens</u>.--In evaluating the strength properties of the various soil-water-asphalt combinations, three different variables were used. These were drying time before compaction, curing time after compaction, and soaking time after compaction. The drying times used were 0, 3, or 6 days while the curing and soaking periods were 0 or 3 days.

The compressive strength was determined for each combination of Soil XI plus MC-2 cutback asphalt, utilizing the optimum water content and the corresponding dry density.

The compressive strength was taken as the triaxial shear strength for a confining pressure of 20 pounds per square inch.

Three samples, 2.8 inches in diameter and 5.6 inches high, were molded for each combination of soil, water content, asphalt content, drying time and curing time. For samples to be soaked, only two were made for each unique combination.

The procedure used for preparing the strength-test specimens was as follows:

1. A calculation was made of the amount of each ingredient necessary to yield a batch sufficient for three molded specimens at the maximum dry density and optimum moisture content. The calculated batch weights were then increased enough to provide two moisture samples of about 100 grams each.
2. The adjusted amount of each ingredient was weighed to the nearest 0.01 pound and the constituents blended and mixed in the same manner as used for the moisture-density mixes.

For mixes that were not to be dried before compaction, molding of the specimens was begun as soon after completion of the mixing process as possible. The equipment used for molding the specimens is shown in Figures 2 and 3. The procedure followed in the molding and extruding of samples was as follows:

1. The spacers were positioned around the lower piston and the bottom sleeve placed on top of the spacers.
2. About one-third of the weight of mixture necessary for one specimen was placed in the sleeve and rodded 20 times with the tamping rod.
3. The spacers were removed (friction holding the bottom sleeve stationary) and the remainder of the mix needed for this sample placed in the sleeve and rodded 20 times.
4. The bottom piston, with the bottom sleeve containing the mix still stationary, was aligned under the upper piston assembly which was secured to the upper head of the testing machine (Figure 3).

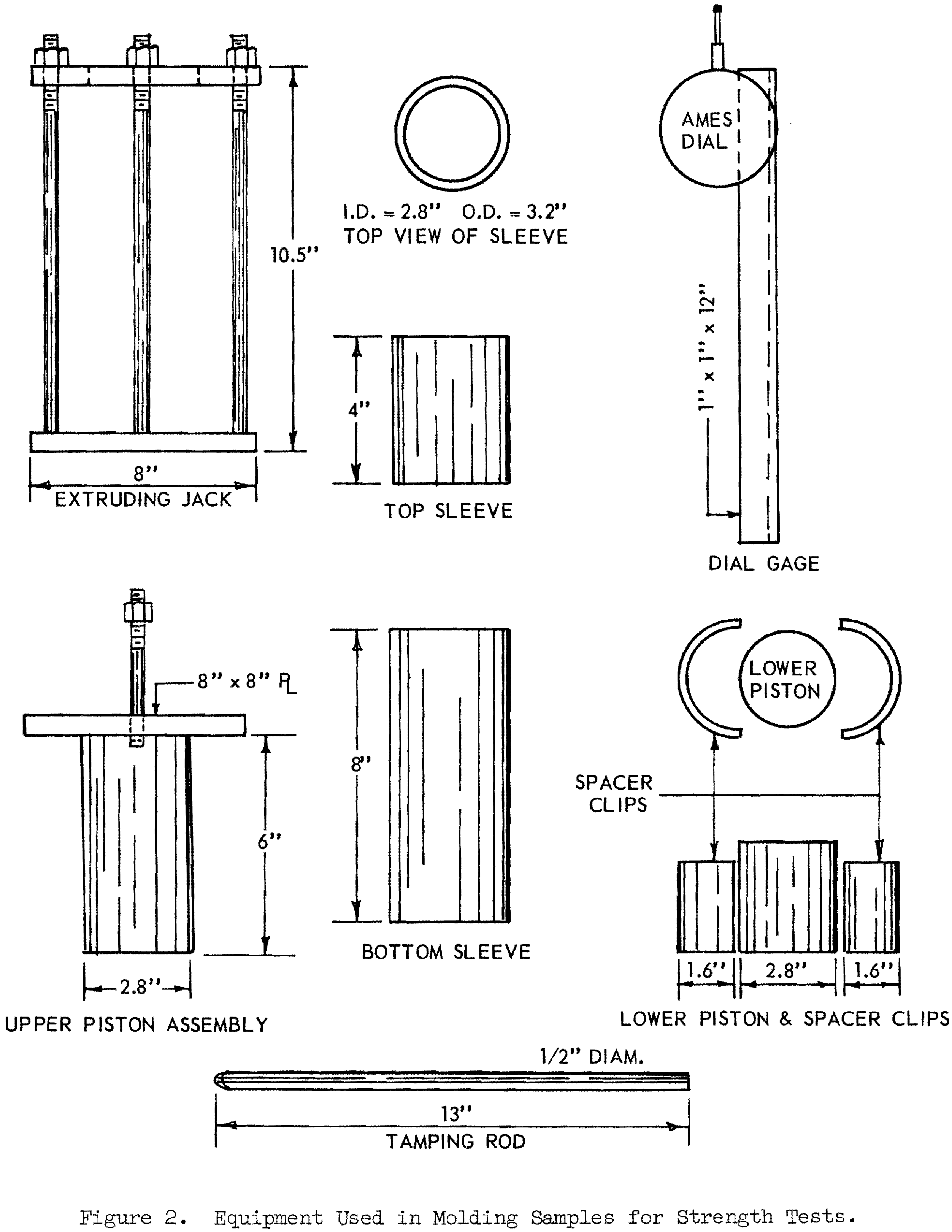

Figure 2. Equipment Used in Molding Samples for Strength Tests.

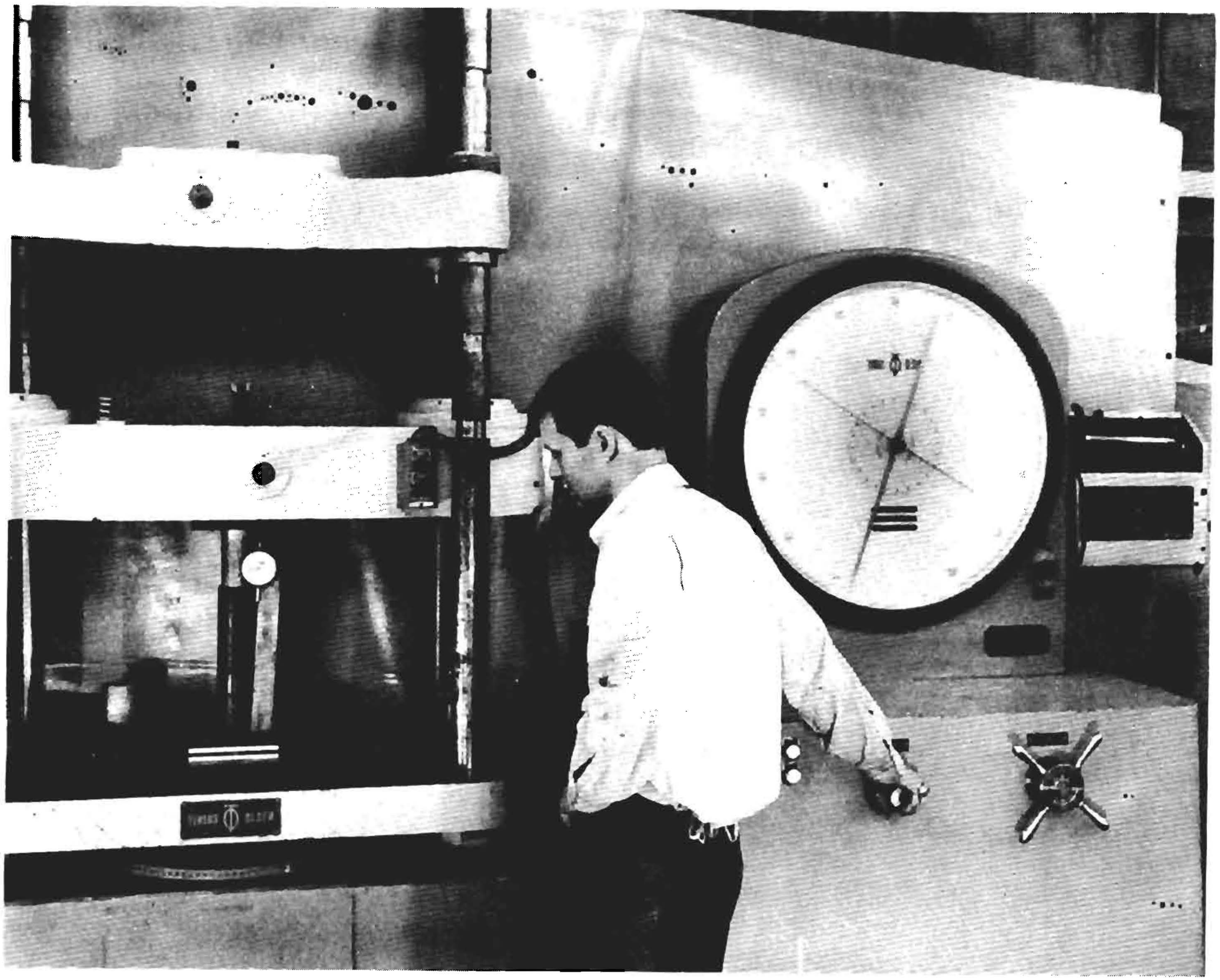

Figure 3. Sample Being Molded for Strength Test.

5. The dial gage assembly was held against the bottom sleeve and the sample compacted at a rate of strain of 0.035 inch per minute.
6. When the dial gage reading was such that it corresponded to a height of sample of 5.6 inches, the machine was immediately stopped. Allowance was made for rebound of the sample, the exact amount depending on the type of mix being compacted.
7. The compacted sample was extruded from the bottom sleeve with the use of a manually operated extruding jack.
8. The sample height and weight were measured and the sample immediately sealed in a polyethylene freezer-bag.

For mixes that were to be dried before compaction, the amount of mix necessary to yield a sample having the desired density was calculated. Depending on whether the samples were to be soaked or not, two or three such quantities were weighed out into tin buckets and allowed to dry for the specified period of time. During the drying period the mix was aerated periodically with a spatula in the same manner that corresponding mixes for moisture-density determinations were aerated. At the completion of the drying period, the mixture was molded using the same procedure given above.

Samples were allowed to cure in the polyethylene bags either for just a few minutes (until they could be prepared for soaking or strength-testing) or for three days.

Following the curing period, each sample was removed from the polyethylene bag and a thin rubber membrane placed over it.

Samples to be soaked were carefully weighed to the nearest 0.01 pound and then placed in the multiple sample, capillary-soak apparatus (see Figure 4). Each sample rested on a porous stone and the top of the sample was covered with

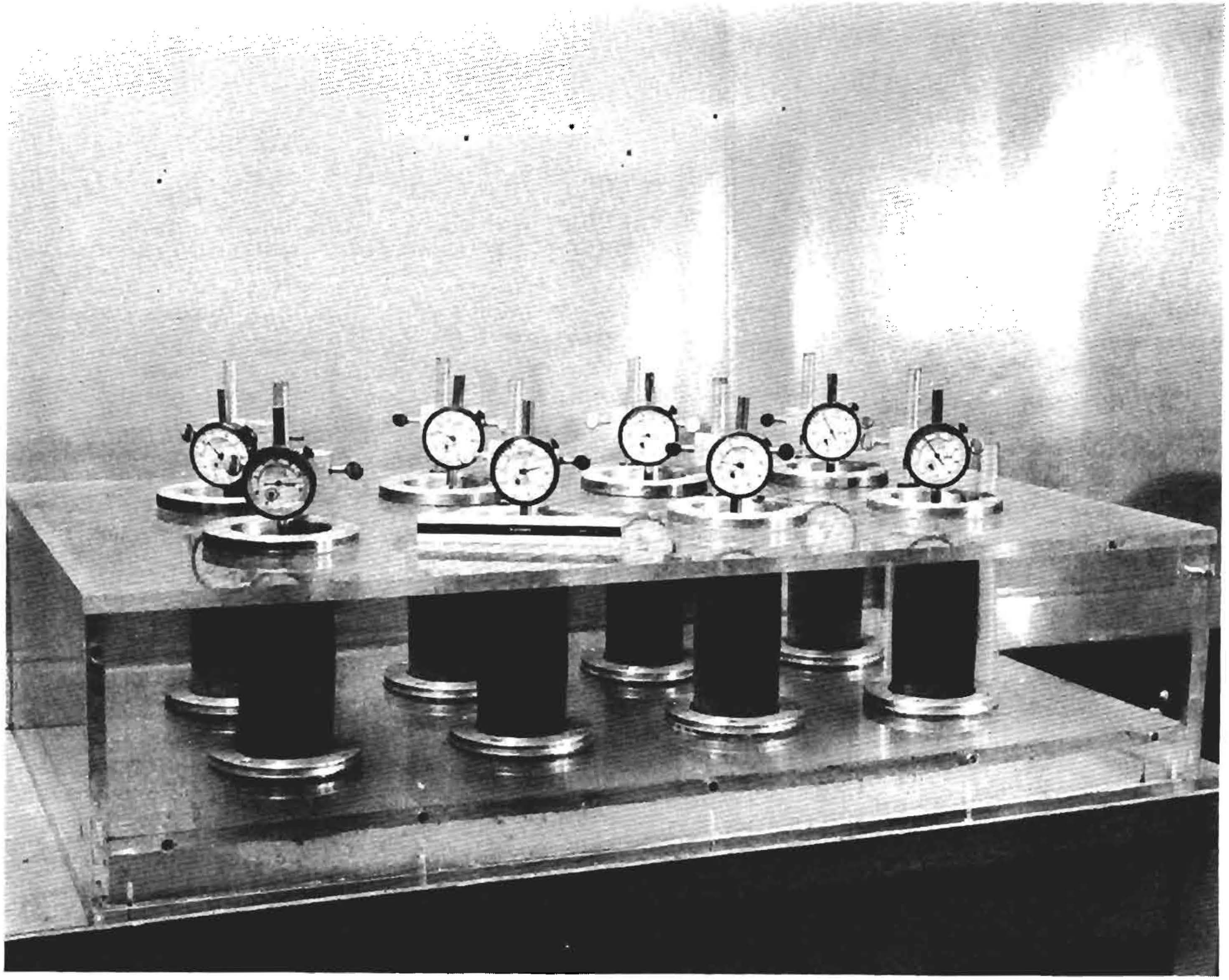

Figure 4. Samples Being Soaked before Triaxial Shear Testing.

a tin cap so that evaporation losses could be neglected and also for the purpose of providing a solid flat surface for the micrometer dial gage stem to rest on. Dial gage readings were recorded at the beginning and end of the 3-day soaking period and the vertical movement computed. After the final dial gage reading was taken, the sample was removed from the soaking device and placed on an absorbent towel for about five seconds in order to remove excess water from the bottom of the sample. The sample was then weighed and immediately placed in the triaxial shear apparatus. The per cent absorption was determined using the following formula:

$$\% \text{ Absorption} = \left(\frac{w_2 - w_1}{w_1}\right) 100\%$$

where w_1 = weight of sample before soaking

w_2 = weight of sample after soaking

The procedure followed in conducting the triaxial shear test was as follows:

1. The sample was placed in the triaxial cell and the top cap placed on the sample; the top of the cell was secured to the lucite cylinder. The shaft was inserted through the top of the cell until it gently rested on the top cap.
2. The triaxial cell was aligned under the upper head of a constant-strain load machine, a dial gage being attached to the upper head. Load was applied at a rate of 0.075 inch per minute of vertical head-travel. A load reading was taken and recorded for each 0.025 inch increment of strain. Figure 5 shows a triaxial shear test in progress.

The compressive strength was taken as the average stress obtained from the failure-load readings of the two or three samples tested. Whenever the value

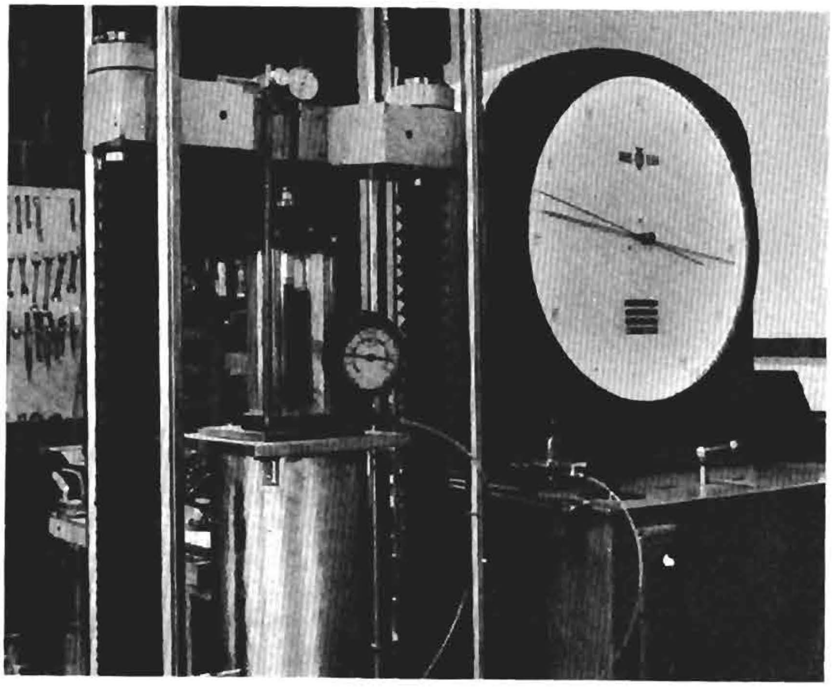

Figure 5. Triaxial Shear Test in Progress.

of compressive strength for one sample varied from that of the other(s) by more than 10 per cent, a new set of samples was molded and tested.

It should be stated here that the soil used was not expansive enough to produce any significant vertical movement; therefore, the results obtained are not reported. It is felt, however, that the method used to measure vertical expansion is adequate for more expansive soils.

CHAPTER V

EVALUATION OF TEST RESULTS

The moisture-density data for Soil XI plus MC-2 were given in Annual Report No. 4 and are repeated here, for convenience, in Table 2 and Figures 6, 7, and 8. The moisture-density relationships for Soil XI plus MC-4, for variable temperature of asphalt at mixing, are given in Table 3 and the appropriate curves shown in Figures 9 through 17. It will be noticed that the latter curves are for water content vs. wet density (not dry density). Because of the great amount of time involved in performing extraction tests, the dry densities were not found. It is expected, however, that by utilizing the relationships between dry and wet densities of the MC-2 mixes a close approximation of the dry densities of the MC-4 mixes could be obtained, if necessary.

Figures 9, 10 and 11 show that, regardless of the temperature of the MC-4 at mixing, the asphalt content producing the highest density is 4 per cent. Also, it is fairly safe to assume that the 4.7 figure under O.M.C. for 150° F is in error and that the actual O.M.C., regardless of temperature, is approximately 5.5 per cent. It is seen that in all cases, except possibly one, the temperature producing the lowest densities is 150° F. This, of course, is also evident in Figures 12 through 17.

Tables 4 through 7 summarize the results of the triaxial shear tests and the absorption tests performed on the samples composed of Soil XI and MC-2 cutback asphalt. These results are presented graphically in Figures 18 through 33 in order to more easily evaluate the effects of the different variables involved.

The importance of a controlled drying time before compaction is evident in Figure 18. At the lower MC-2 asphalt contents the drying time was beneficial to strength, up to 6 days; however, for MC-2 percentages of 4, 5 and 6 the 3-day

TABLE 2

MAXIMUM DRY DENSITIES FOR SOIL XI COMBINED WITH MC-2 CUTBACK ASPHALT, VARYING DRYING PERIOD

	Drying Period (Days)					
	0		3		6	
Per Cent MC-2	M.D.D.	O.M.C.	M.D.D.	O.M.C.	M.D.D.	O.M.C.
0	122.4	9.4	-	-	-	-
1	123.8	7.8	126.3	14.7	124.2	10.6
2	126.1	6.5	126.5	13.6	124.8	10.8
3	124.8	6.0	126.6	12.5	124.9	10.1
4	125.5	4.7	125.5	11.8	124.4	7.7
5	125.2	4.1	125.4	10.9	124.6	6.8
6	125.6	2.8	125.2	7.0	123.9	6.0

TABLE 3

MOISTURE-DENSITY DATA FOR SOIL XI COMBINED WITH MC-4;
VARYING MIXING TEMPERATURE OF MC-4; NO DRYING

	Mixing Temperature of MC-4					
	100° F		125° F		150° F	
Per Cent MC-4	M.W.D.	O.M.C.	M.W.D.	O.M.C.	M.W.D.	O.M.C.
0	133.9	9.4	133.9	9.4	133.9	9.4
1	132.1	8.6	135.2	7.7	134.6	8.1
2	135.7	7.1	135.5	7.1	135.2	6.9
3	136.4	6.3	135.8	6.2	136.6	6.0
4	137.5	5.5	137.5	5.6	136.9	4.7
5	137.1	4.9	137.0	4.7	136.8	5.0
6	136.8	3.9	136.8	4.0	136.3	3.2

TABLE 4

SUMMARY OF COMPRESSIVE STRENGTH TEST RESULTS
(NO CURING, NO SOAKING)

MC-2 Content (%)	Optimum Water Content (%)	Days of Drying Back	Days of Curing	Days of Soaking	Maximum Dry Density (lb./cu. ft.)	Normal Stress σ_1, psi (at σ_3=20 psi)
0	9.4	0	0	0	122.4	60
1	7.8	0	0	0	123.8	79
1	10.6	3	0	0	124.2	86
1	14.7	6	0	0	126.3	88
2	6.5	0	0	0	126.1	85
2	10.8	3	0	0	124.8	88
2	13.6	6	0	0	126.5	97
3	6.0	0	0	0	124.8	83
3	10.1	3	0	0	124.9	86
3	12.5	6	0	0	126.6	86
4	4.7	0	0	0	125.5	86
4	6.8	3	0	0	124.6	86
4	11.8	6	0	0	125.5	72
5	4.1	0	0	0	125.2	86
5	7.7	3	0	0	124.4	87
5	10.9	6	0	0	125.4	78
6	2.8	0	0	0	125.6	87
6	6.0	3	0	0	123.9	87
6	7.0	6	0	0	125.2	82

TABLE 5

SUMMARY OF COMPRESSIVE STRENGTH TEST RESULTS
(NO CURING, 3 DAYS OF SOAKING)

MC-2 Content (%)	Optimum Water Content (%)	Days of Drying Back	Days of Curing	Days of Soaking	Maximum Dry Density (lb./cu. ft.)	Normal Stress σ_1, psi (at σ_3=20 psi)	% Absorpt $\frac{w_2-w_1}{w_1}$x10
0	9.4	0	0	3	122.4	27	1.8
1	7.8	0	0	3	123.8	23	3.3
1	10.6	3	0	3	124.2	26	6.6
1	14.7	6	0	3	126.3	28	6.7
2	6.5	0	0	3	126.1	32	3.1
2	10.8	3	0	3	124.8	34	5.7
2	13.6	6	0	3	126.5	33	4.8
3	6.0	0	0	3	124.8	50	3.4
3	10.1	3	0	3	124.9	52	3.1
3	12.5	6	0	3	126.6	67	1.1
4	4.7	0	0	3	125.5	50	2.9
4	6.8	3	0	3	124.6	66	2.0
4	11.8	6	0	3	125.5	78	0.1
5	4.1	0	0	3	125.2	63	1.3
5	7.7	3	0	3	124.4	72	2.8
5	10.9	6	0	3	125.4	73	0.0
6	2.8	0	0	3	125.6	52	0.6
6	6.0	3	0	3	123.9	74	0.6
6	7.0	6	0	3	125.2	76	0.0

TABLE 6

SUMMARY OF COMPRESSIVE STRENGTH TEST RESULTS
(3 DAYS OF CURING, NO SOAKING)

MC-2 Content (%)	Optimum Water Content (%)	Days of Drying Back	Days of Curing	Days of Soaking	Maximum Dry Density (lb./cu. ft.)	Normal Stress σ_1, psi (at σ_3=20 psi)
0	9.4	0	3	0	122.4	72
1	7.8	0	3	0	123.8	79
1	10.6	3	3	0	124.2	84
1	14.7	6	3	0	126.3	85
2	6.5	0	3	0	126.1	86
2	10.8	3	3	0	124.8	84
2	13.6	6	3	0	126.5	93
3	6.0	0	3	0	124.8	70
3	10.1	3	3	0	124.9	89
3	12.5	6	3	0	126.6	83
4	4.7	0	3	0	125.5	81
4	6.8	3	3	0	124.6	83
4	11.8	6	3	0	125.5	79
5	4.1	0	3	0	125.2	89
5	7.7	3	3	0	124.4	85
5	10.9	6	3	0	125.4	74
6	2.8	0	3	0	125.6	78
6	6.0	3	3	0	123.9	83
6	7.0	6	3	0	125.2	68

TABLE 7

SUMMARY OF COMPRESSIVE STRENGTH TEST RESULTS
(3 DAYS OF CURING, 3 DAYS OF SOAKING)

MC-2 Content (%)	Optimum Water Content (%)	Days of Drying Back	Days of Curing	Days of Soaking	Maximum Dry Density (lb./cu. ft.)	Normal Stress σ_1, psi (at σ_3=20 psi)	% Absorption $\frac{w_2-w_1}{w_1}$x100%
0	9.4	0	3	3	122.4	64	2.0
1	7.8	0	3	3	123.8	38	3.2
1	10.6	3	3	3	124.2	10	7.7
1	14.7	6	3	3	126.3	70	0.7
2	6.5	0	3	3	126.1	52	3.8
2	10.8	3	3	3	124.8	8	6.3
2	13.6	6	3	3	126.5	94	0.3
3	6.0	0	3	3	124.8	19	3.2
3	10.1	3	3	3	124.9	81	0.0
3	12.5	6	3	3	126.6	87	0.0
4	4.7	0	3	3	125.5	33	3.0
4	6.8	3	3	3	124.6	89	0.2
4	11.8	6	3	3	125.5	80	0.0
5	4.1	0	3	3	125.2	66	0.5
5	7.7	3	3	3	124.4	63	2.7
5	10.9	6	3	3	125.4	71	0.0
6	2.8	0	3	3	125.6	69	0.7
6	6.0	3	3	3	123.9	46	1.3
6	7.0	6	3	3	125.2	84	0.2

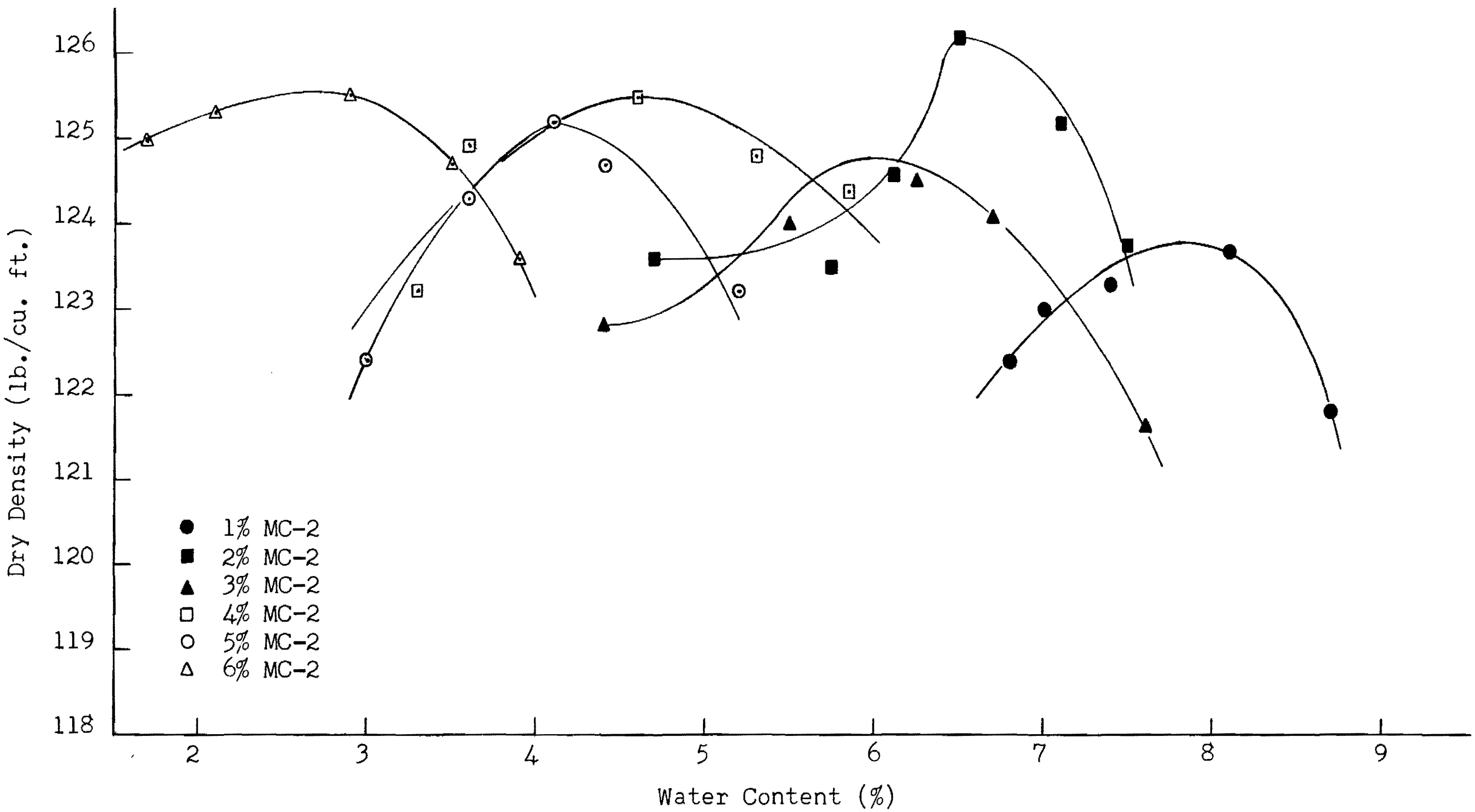

Figure 6. Moisture-Density Curves for Soil XI Plus MC-2, No Drying.

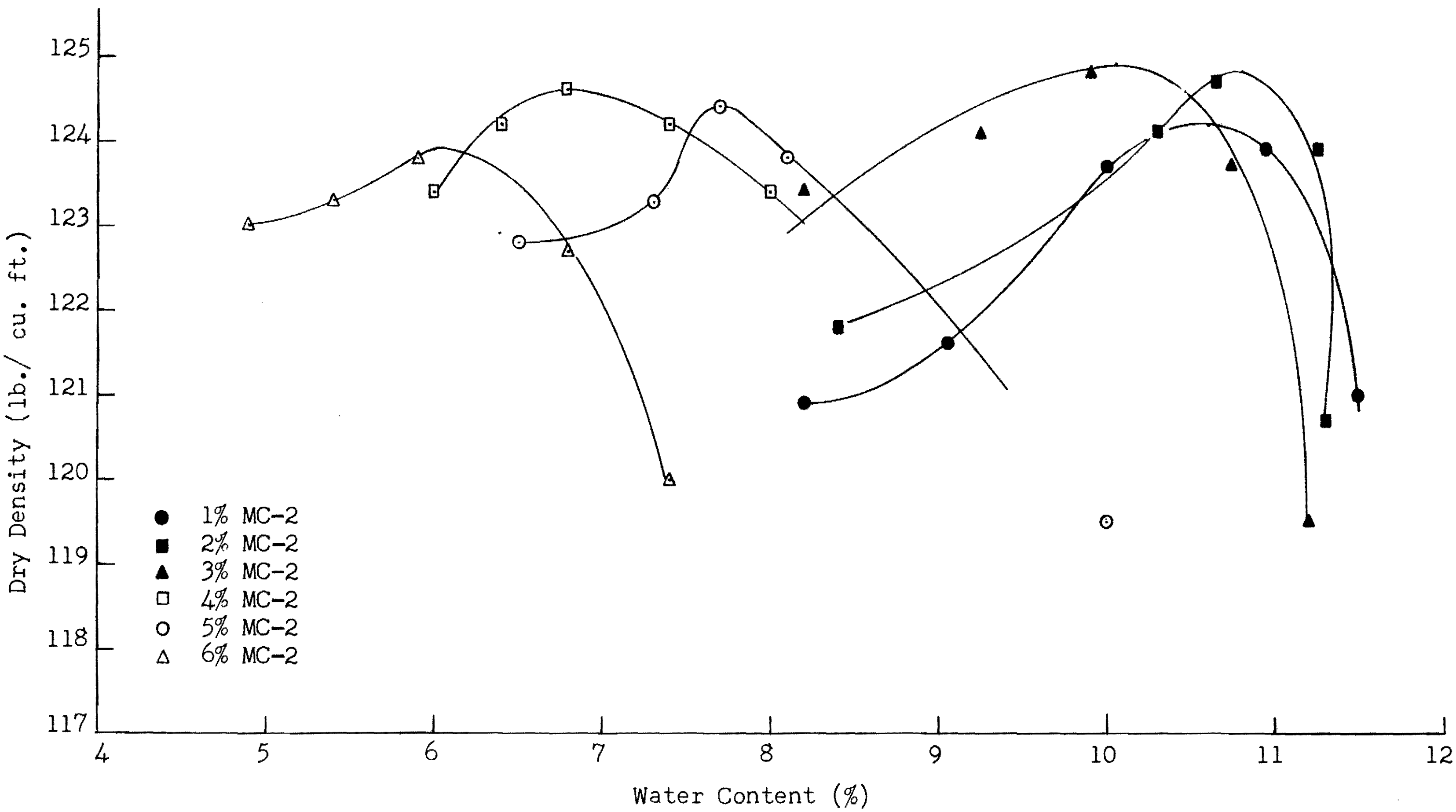

Figure 7. Moisture-Density Curves for Soil XI Plus MC-2, 3 Days of Drying.

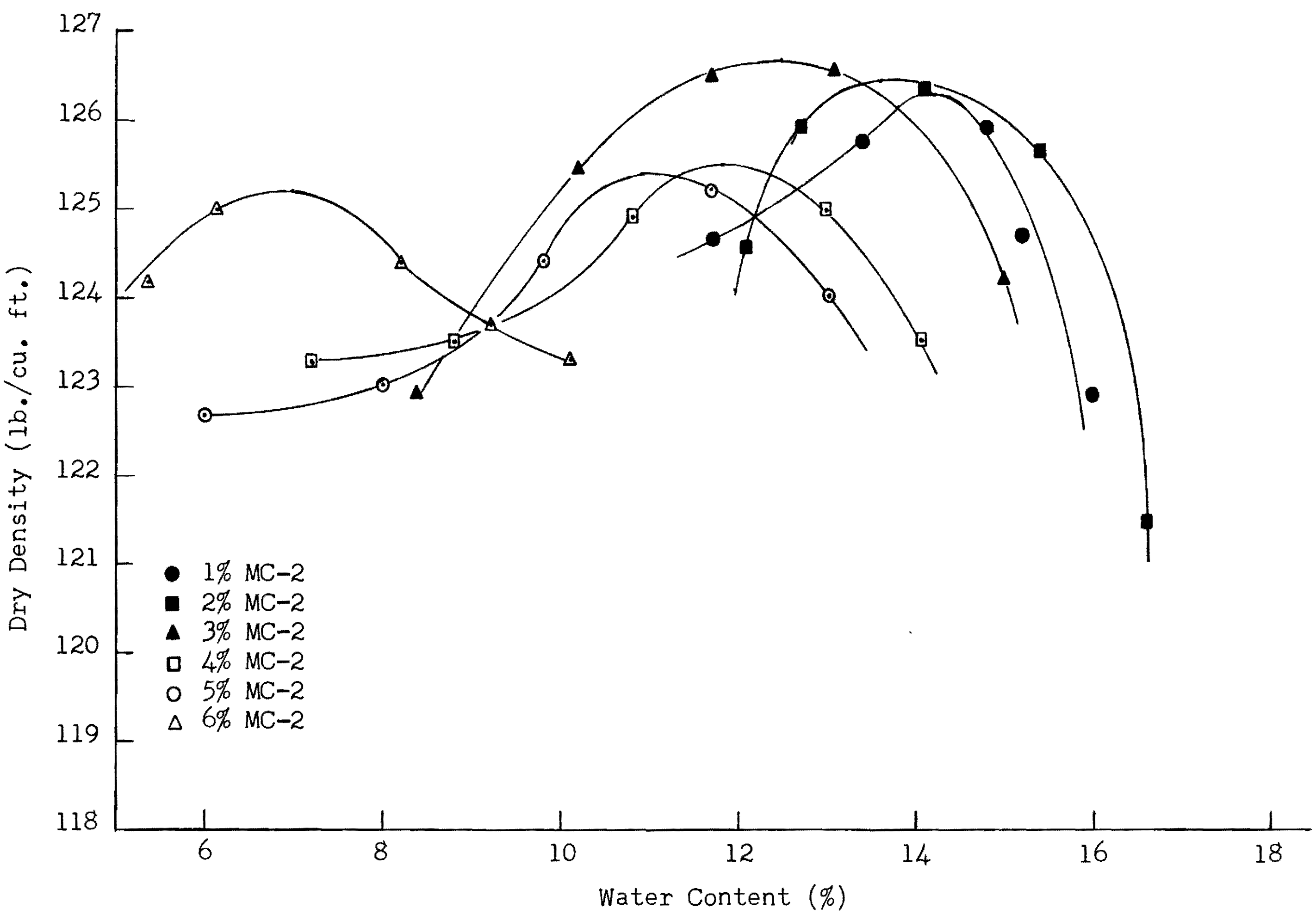

Figure 8. Moisture-Density Curves for Soil XI Plus MC-2, 6 Days of Drying.

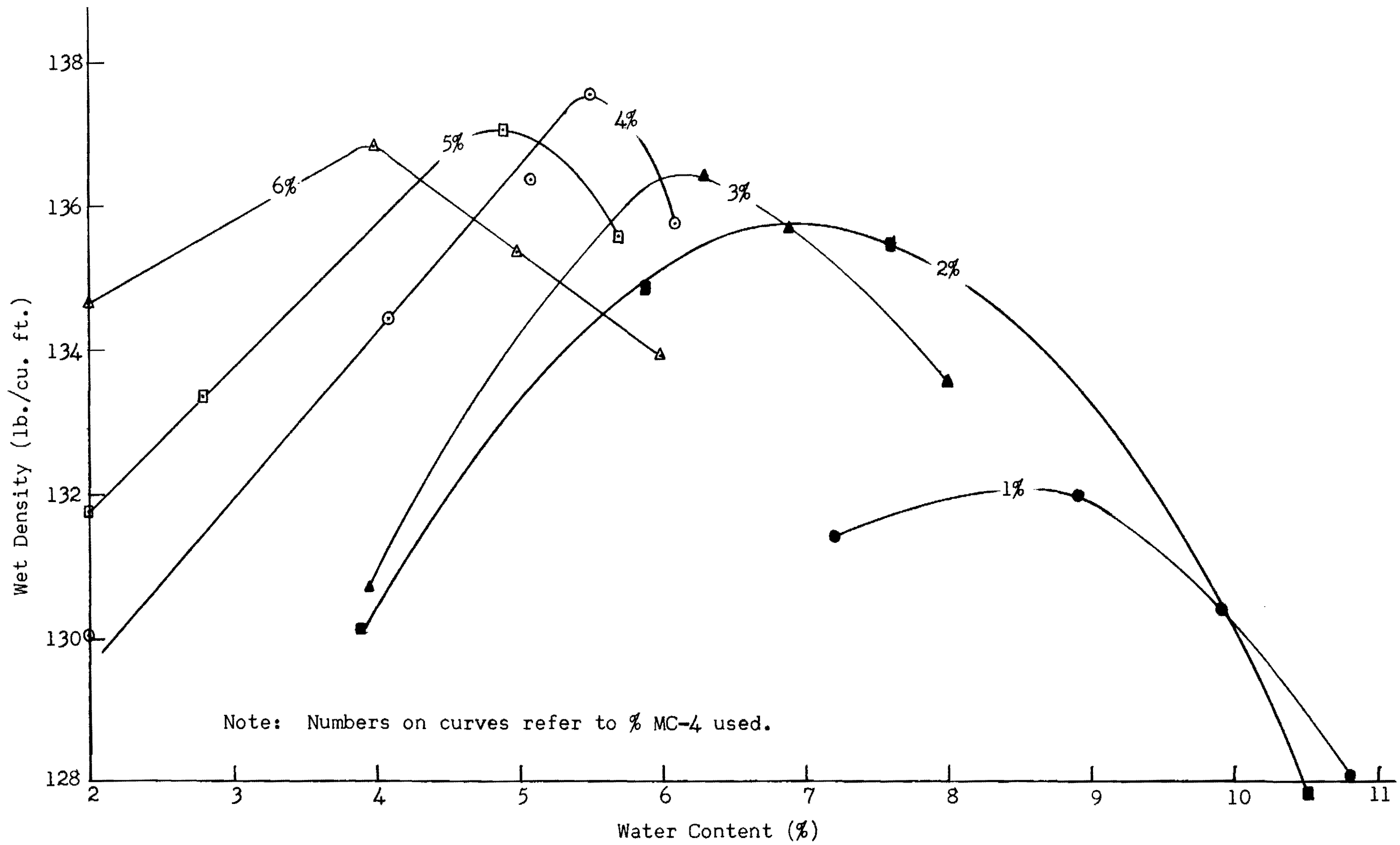

Figure 9. Moisture-Density Curves for Soil XI Plus MC-4 at 100°F (No Drying).

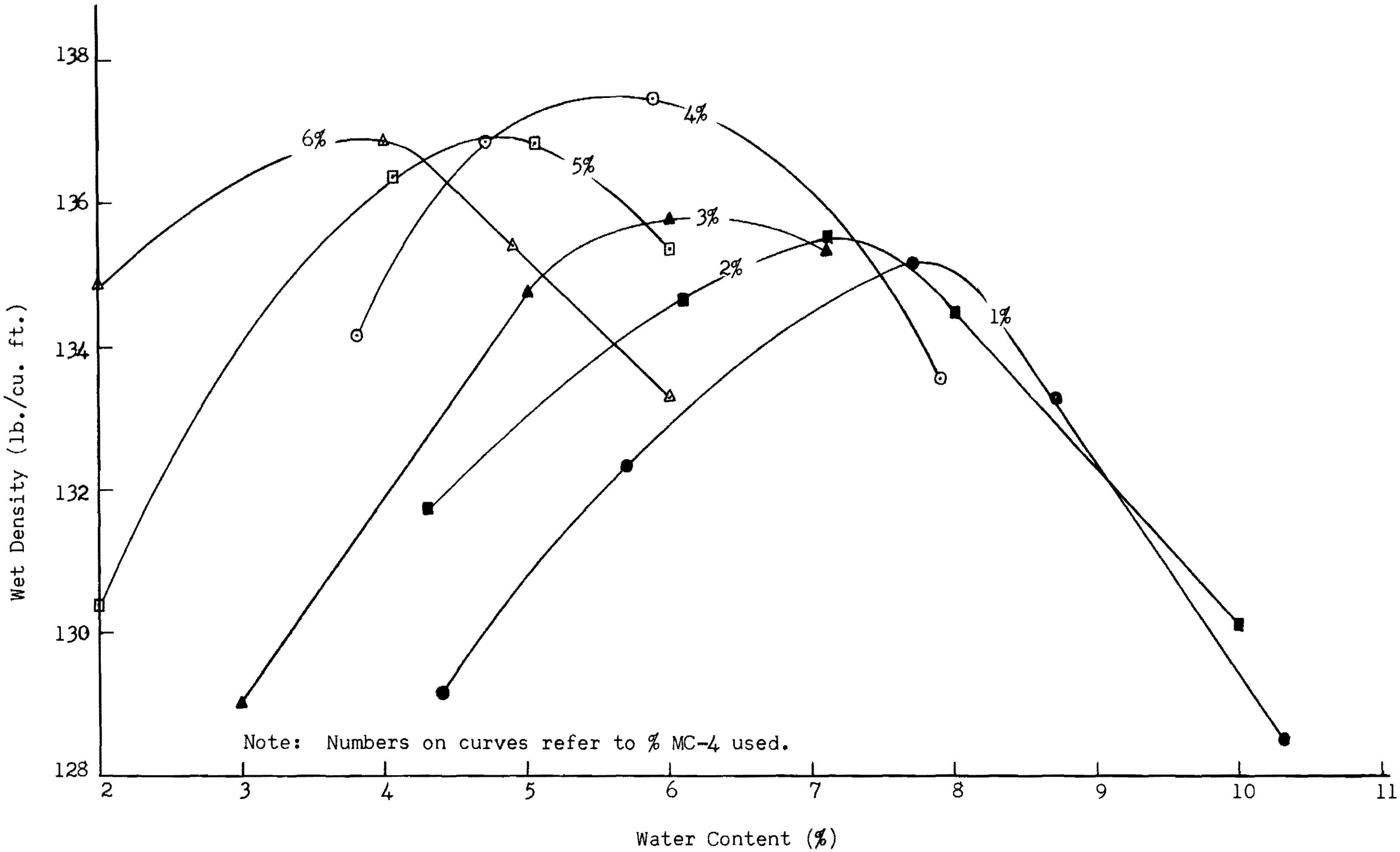

Figure 10. Moisture-Density Curves for Soil XI Plus MC-4 at 125°F (No Drying).

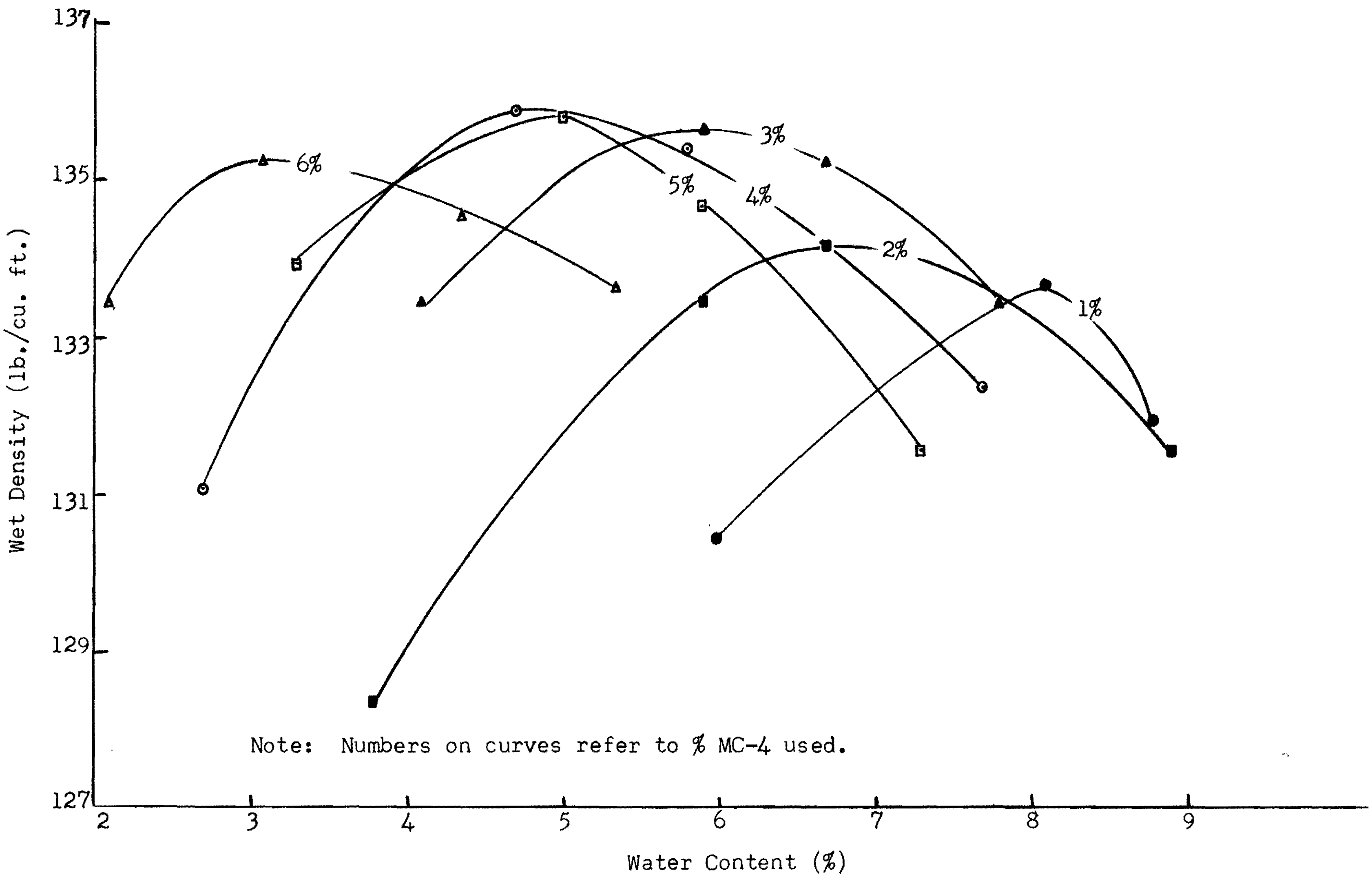

Figure 11. Moisture-Density Curves for Soil XI Plus MC-4 at 150°F (No Drying).

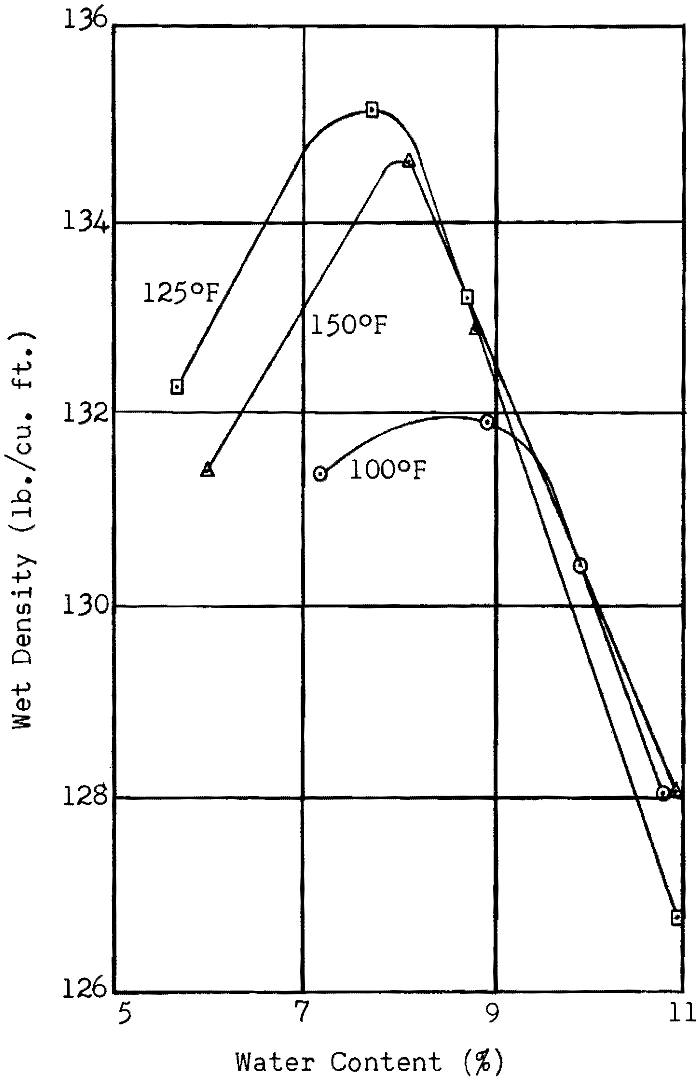

Figure 12. Moisture-Density Curves for Soil XI Plus 1% MC-4 (variable temperature).

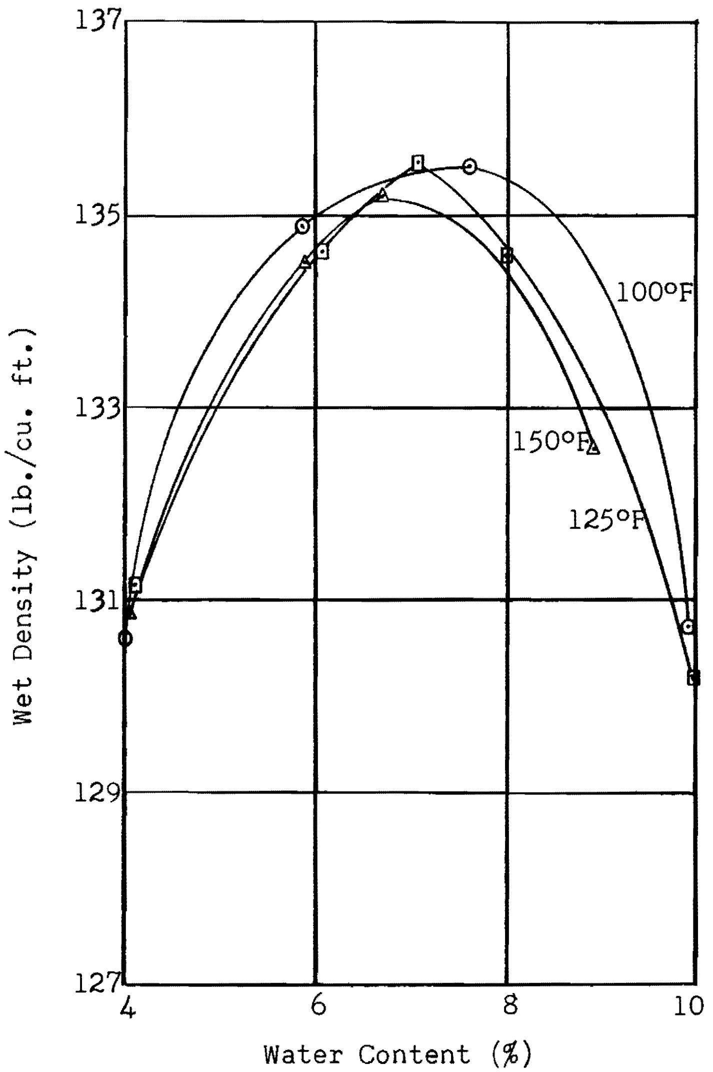

Figure 13. Moisture-Density Curves for Soil XI Plus 2% MC-4 (variable temperature).

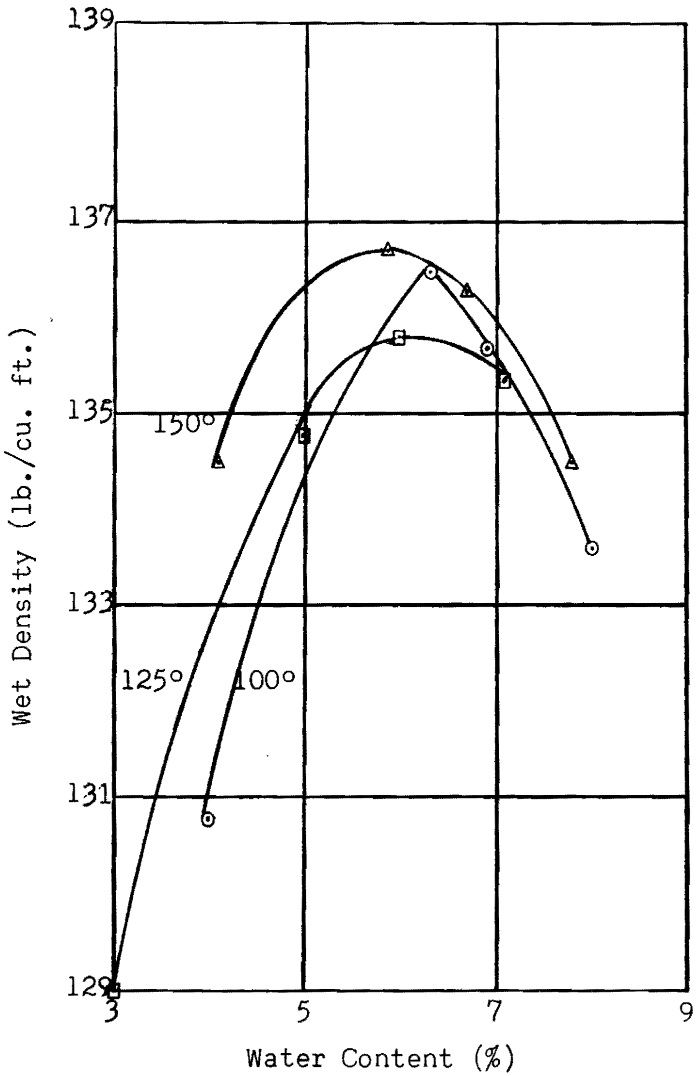

Figure 14. Moisture-Density Curves for Soil XI Plus 3% MC-4 (variable temperature).

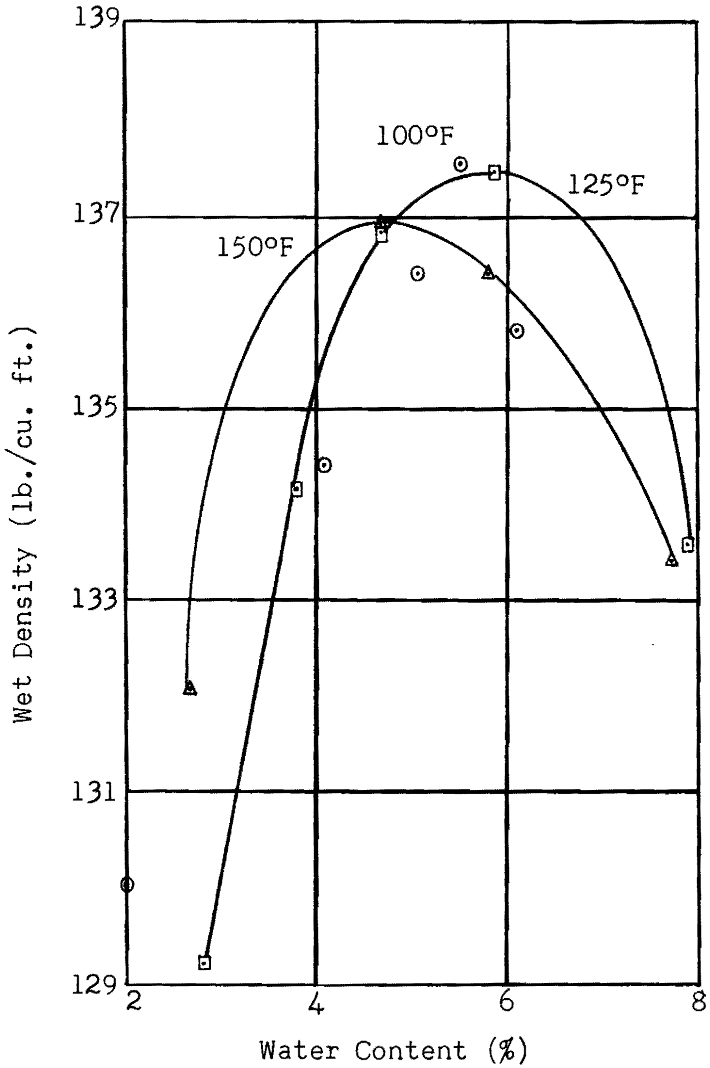

Figure 15. Moisture-Density Curves for Soil XI Plus 4% MC-4 (variable temperature).

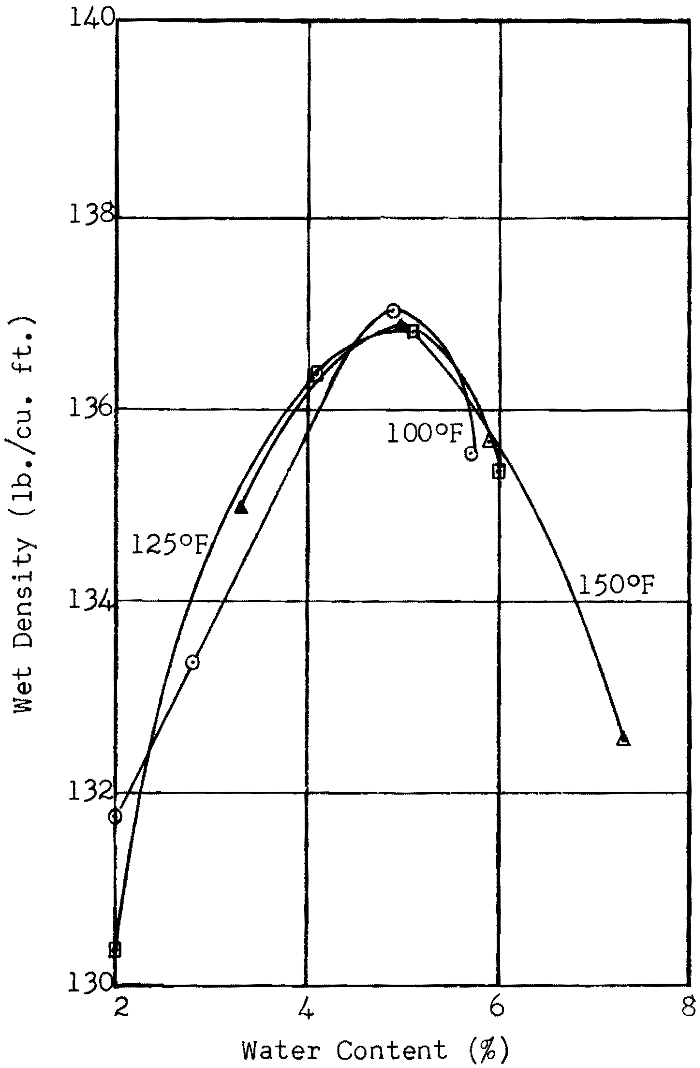

Figure 16. Moisture-Density Curves for Soil XI Plus 5% MC-4 (variable temperature).

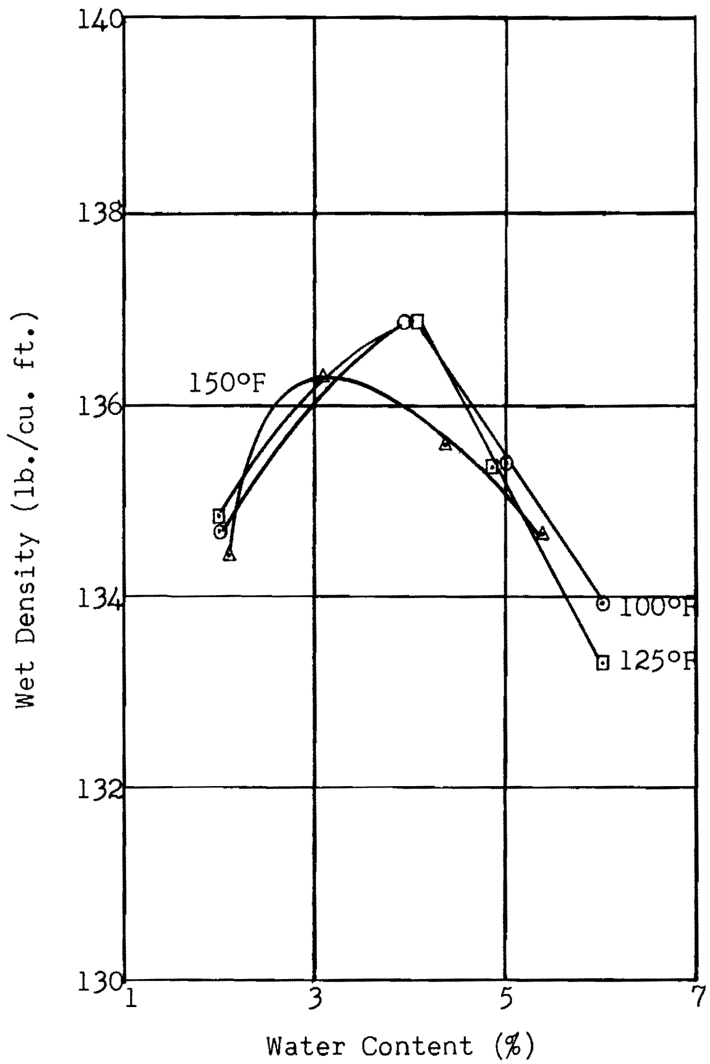

Figure 17. Moisture-Density Curves for Soil XI Plus 6% MC-4 (variable temperature).

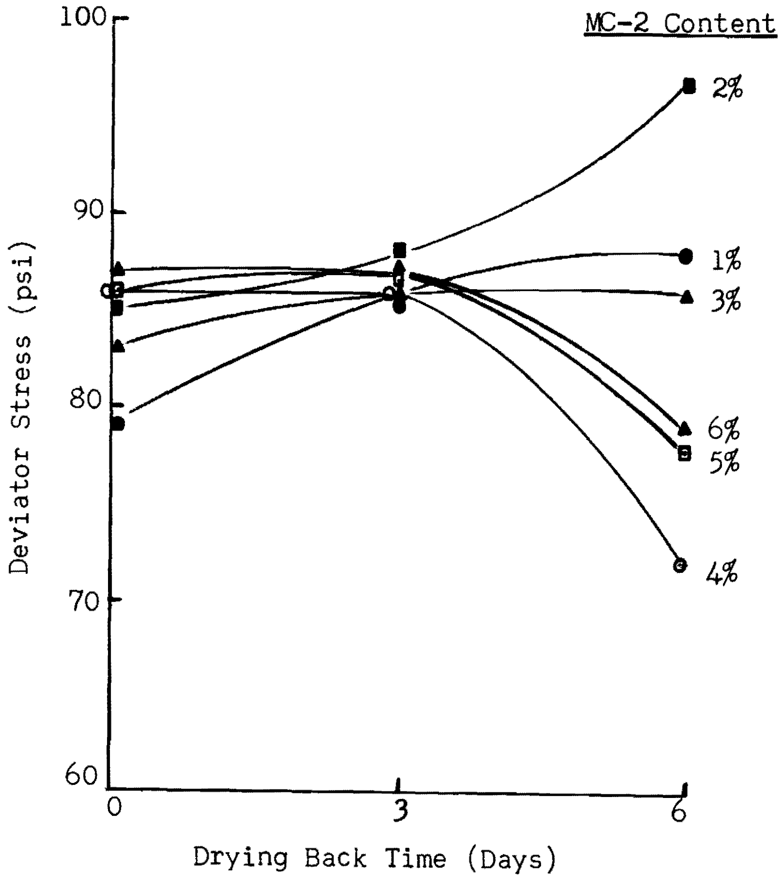

Figure 18. Effect of Drying-back on Strength (No Curing, No Soaking).

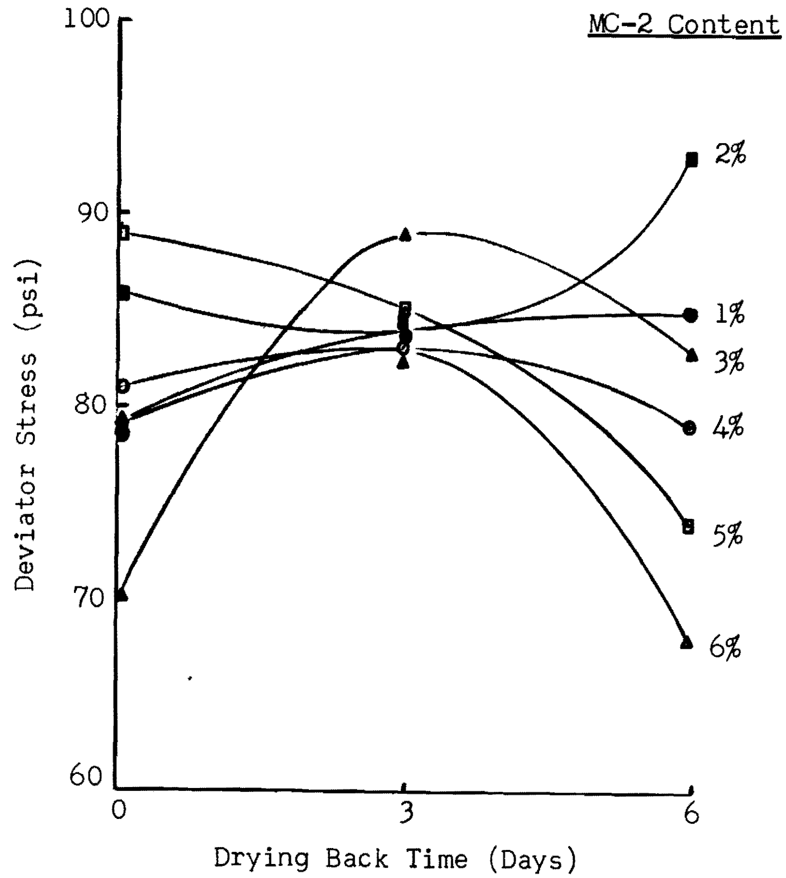

Figure 19. Effect of Drying-back on Strength (3 Days of Curing, No Soaking).

drying period was of little benefit while 6 days of drying was actually detrimental to strength. The same general trend is true for samples subjected to 3 days of curing before testing (see Figure 19). For samples which were not cured but soaked prior to testing, the drying period had much more influence on preserving strength at the higher MC-2 contents than at the lower ones. This is shown in Figure 20 and may indicate that the waterproofing properties of MC-2 cutback asphalt can be enhanced by allowing time for the individual soil grains to become coated with asphalt. The accompanying graph (Figure 20a) substantiates this by showing that the samples at the three higher MC-2 contents had decreasing water absorption with increasing drying time, to the extent that there was no absorption at all for the mixes dried 6 days before compaction.

Figures 21 and 21a were intended for the same purpose as were Figures 20 and 20a, the difference being in the curing time. The results obtained were not consistent with the latter graphs and are difficult to explain. However, Figure 21a does support the data of Figure 21; that is, it shows that only two asphalt contents, 3 and 4 per cent, were effective with 3 days of drying back.

The effect of 3 days of soaking on samples compacted immediately after mixing can be found from Figures 22 and 22a. The relative effect on strength can be determined from the slopes of the lines. It is surprising to see that the strength of a sample containing 1 or 2 per cent MC-2 decreases at a faster rate than that of a sample containing no MC-2. Also, it required at least 5 per cent asphalt to decrease the amount of water absorbed by a sample.

The graphs of Figures 23 and 23a illustrate that, for samples dried 3 days, the waterproofing effect of MC-2 increases with increase in its content. Comparing these results with those of Figures 22 and 22a seems to point out that, as previously noted, the soil-water-asphalt system passes through a certain state

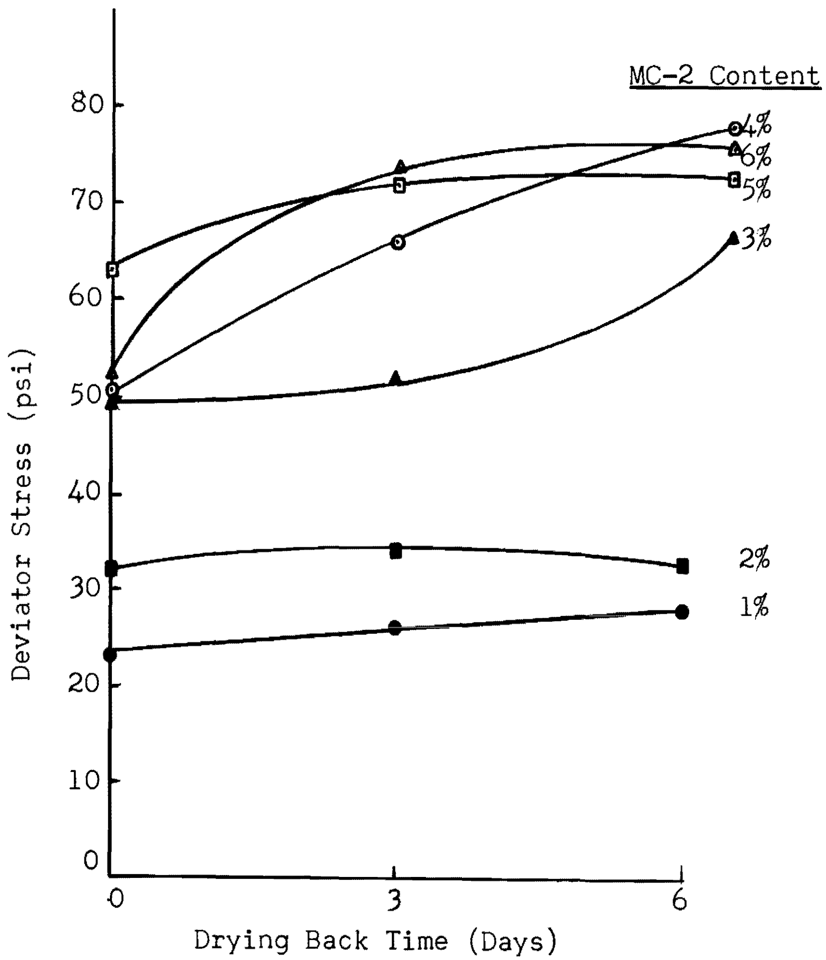

Figure 20. Effect of Drying-Back on Strength (No Curing, 3 Days of Soaking).

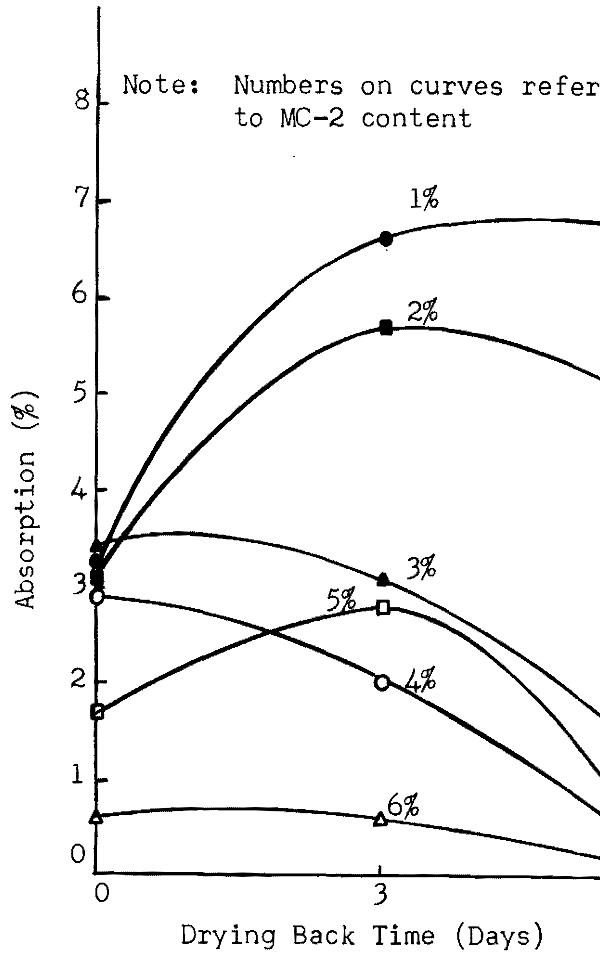

Figure 20a. Effect of Drying-back on Absorption (No Curing, 3 Days of Soaking).

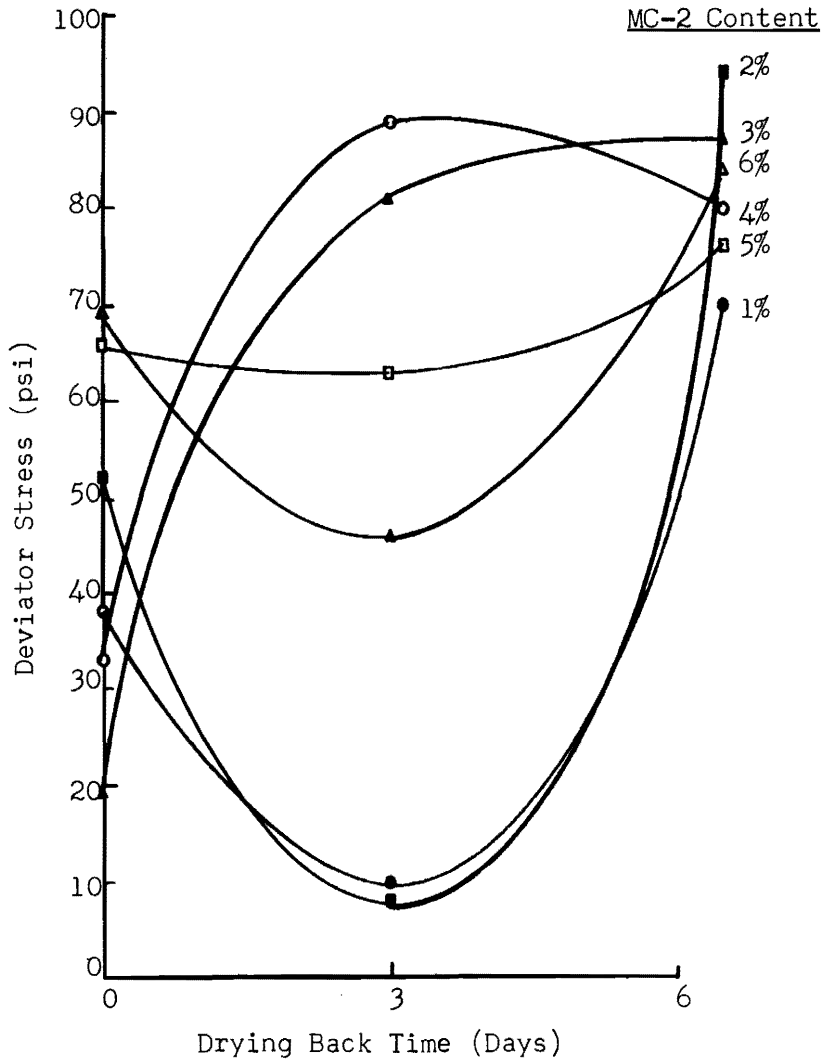

Figure 21. Effect of Drying-back on Strength (3 Days of Curing, 3 Days of Soaking).

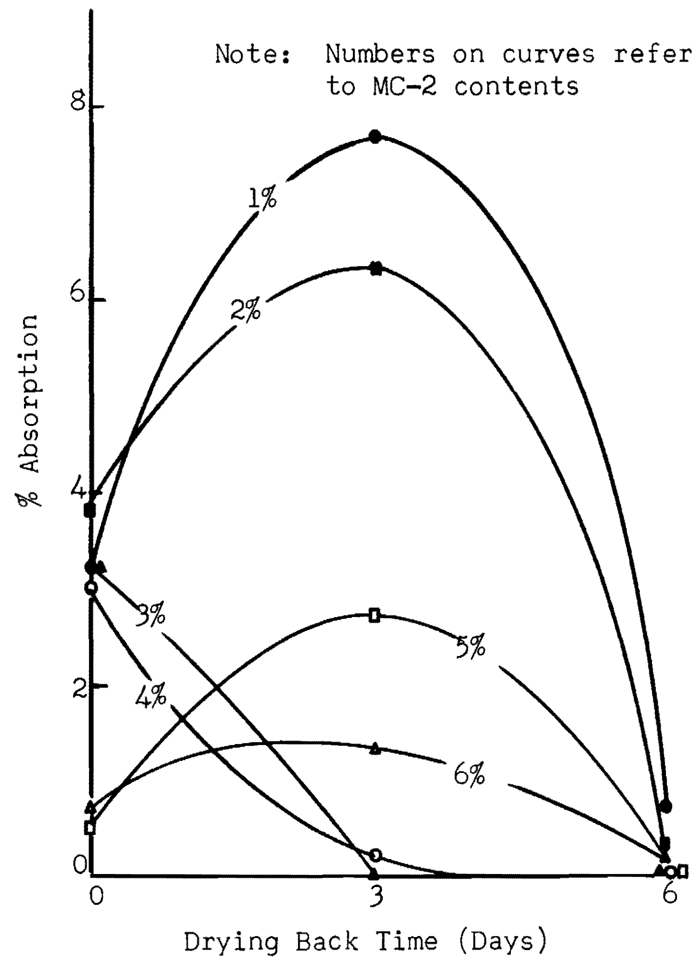

Figure 21a. Effect of Drying-back on Absorption (3 Days of Curing, 3 Days of Soaking).

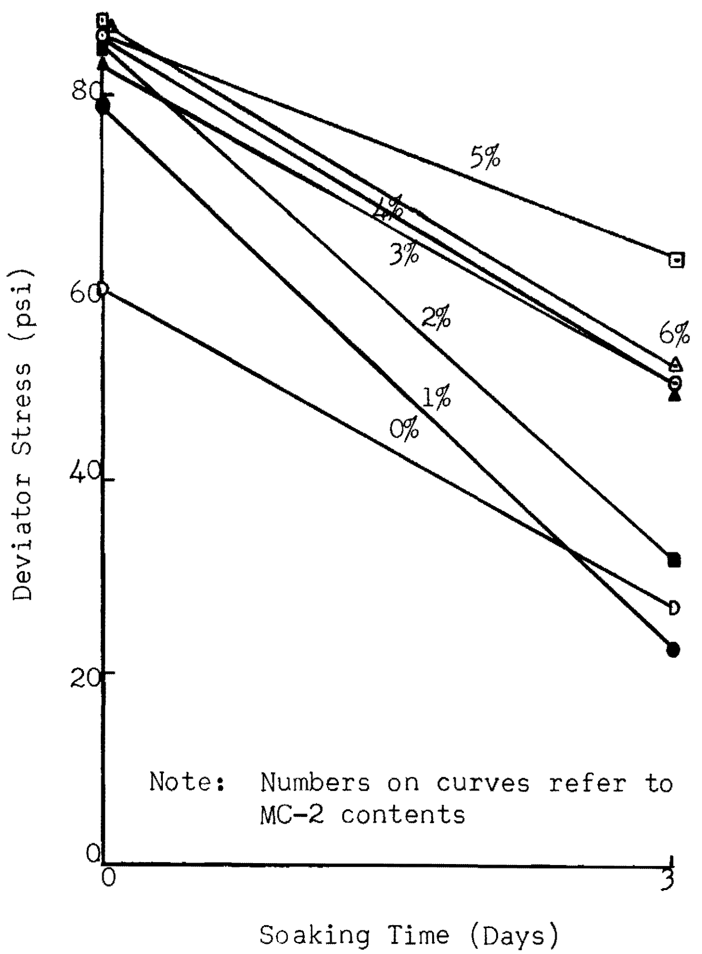

Figure 22. Effect of Soaking on Strength (No Drying-back, No Curing).

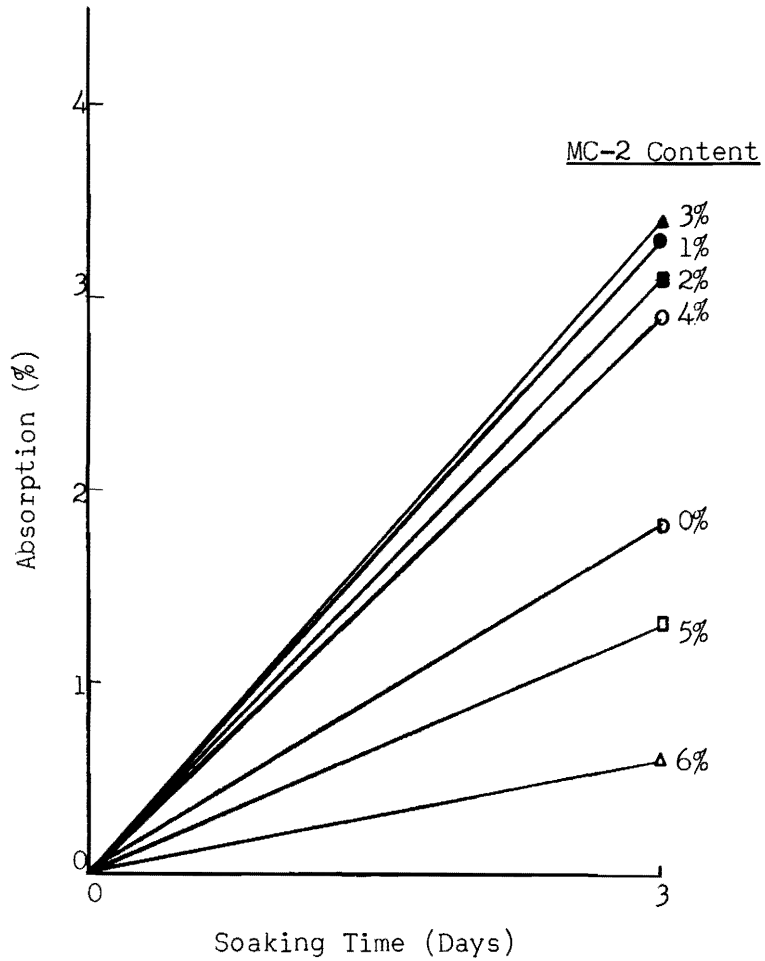

Figure 22a. Effect of MC-2 Content on Absorption (No Drying-back, No Curing).

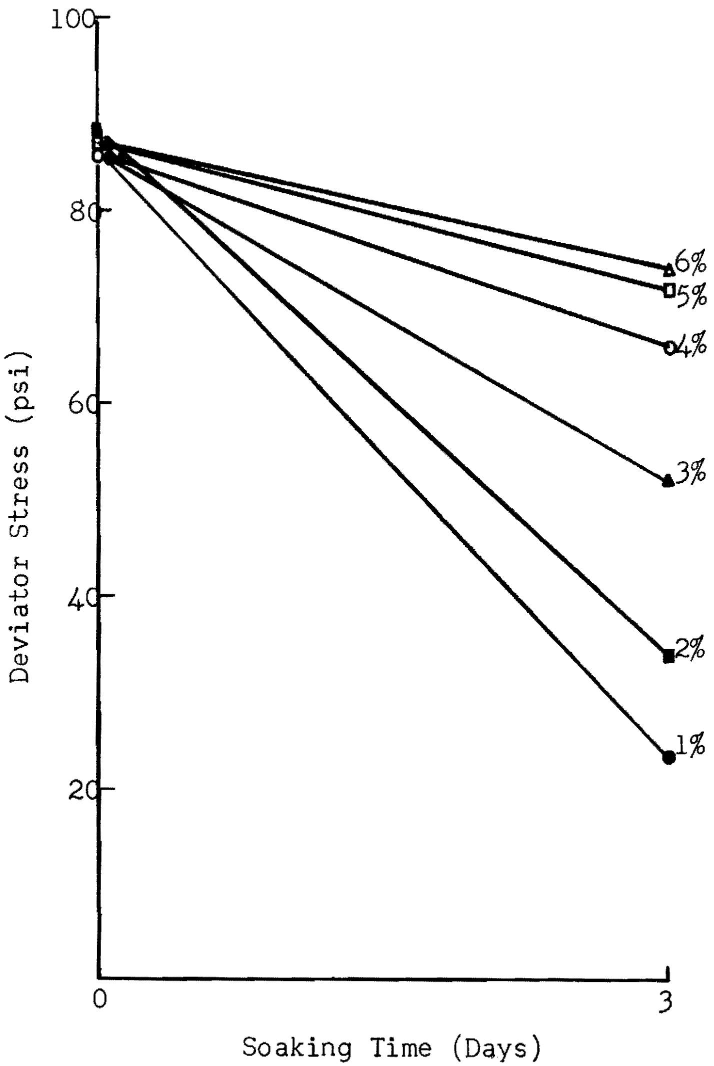

Figure 23. Effect of Soaking on Strength (3 Days of Drying-back, No Curing).

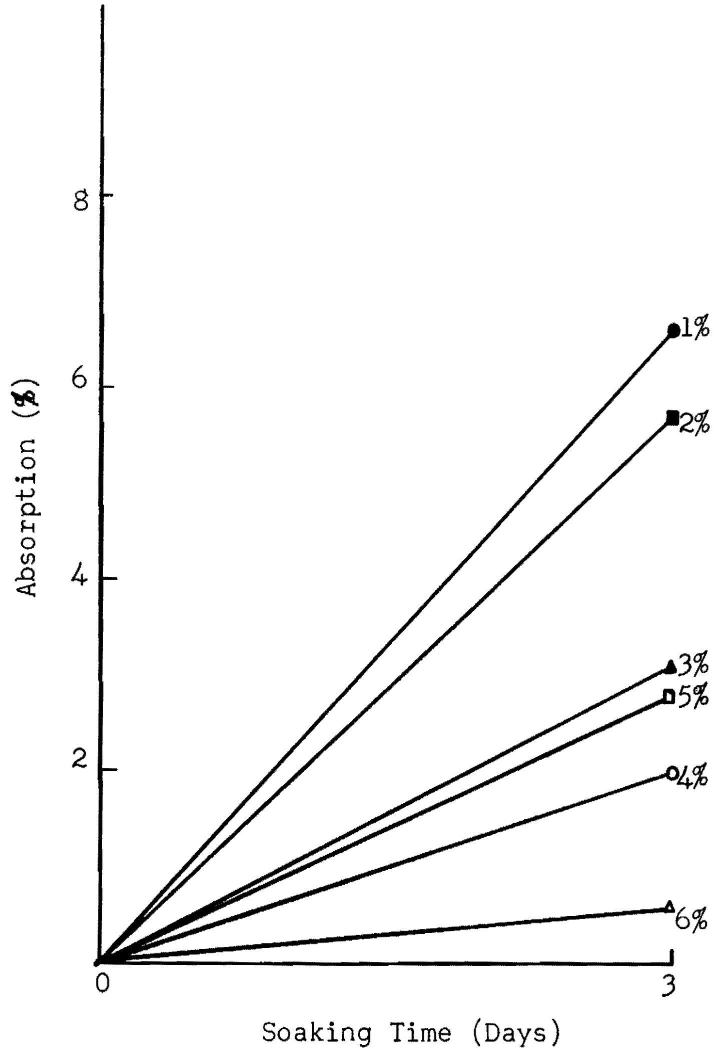

Figure 23a. Effect of MC-2 Content on Absorption (3 Days of Drying-back, No Curing).

in which it is most stable. This is still further substantiated by the graphs of Figures 24 and 24a which show that, with the exception of the mixes containing 5 or 6 per cent MC-2, the strength was less affected by soaking as the asphalt content was increased. These two exceptions may be explained by reasoning that the greater the amount of asphalt used the longer the period of asphalt distribution necessary for maximum effective waterproofness of the soil-water-asphalt system.

Figures 25 and 25a show the effect of 3 days of soaking on samples compacted immediately after mixing and cured for 3 days. As in Figures 22 and 22a, previously discussed, the effect on strength can be found from the slopes of the lines. Samples containing up to 4 per cent MC-2 were affected more than samples containing no asphalt at all. Again, the companion graph (in this case Figure 25a) substantiated this fact by showing that only the lines for 5 or 6 per cent MC-2 have flatter slopes than the zero per cent line.

Figures 26 and 26a both indicate that 3 and 4 per cent are the best asphalt contents to use for 3 days of drying and 3 days of curing. Again, it can only be surmised that the mixes containing 5 and 6 per cent MC-2 had not yet reached their states of optimum waterproofness.

The reasoning that advocates an optimum state for waterproofness can be tested if the effects of longer drying times are studied. In Figures 27 and 27a, the results of tests conducted on samples dried for 6 days before compaction are shown. It is seen that now the 5 and 6 per cent lines have flattened considerably, thus indicating that each of these systems has reached a state which allows it to be effective in maintaining strength under the normally detrimental effect of capillary soaking.

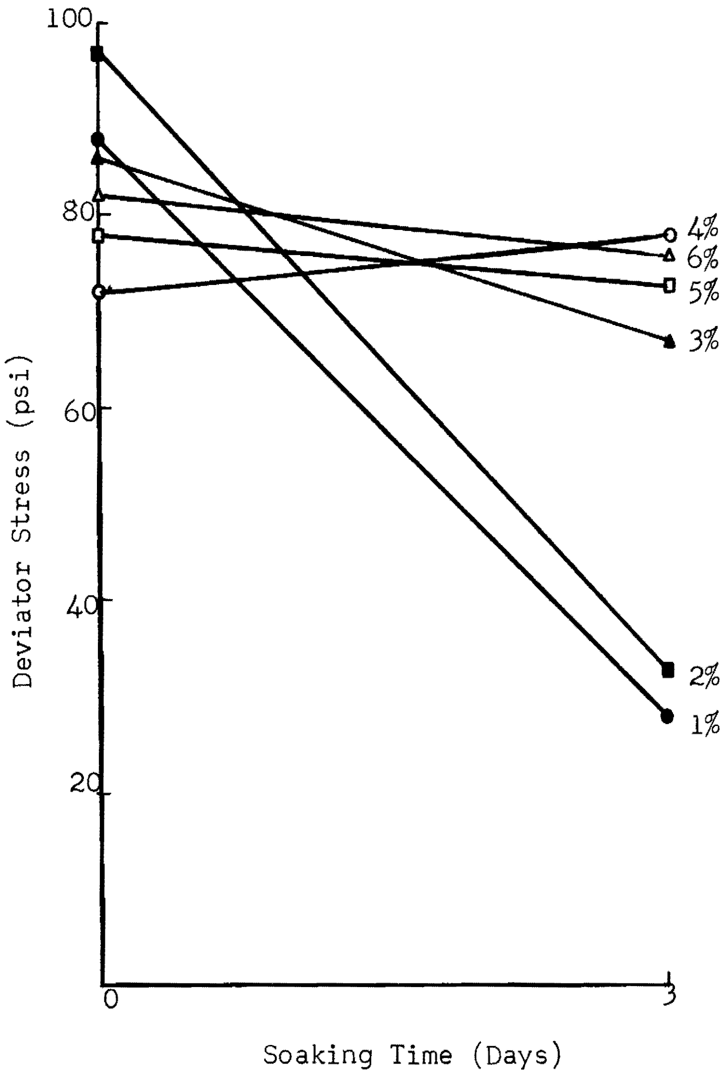

Figure 24. Effect of Soaking on Strength (6 Days of Drying-back, No Curing).

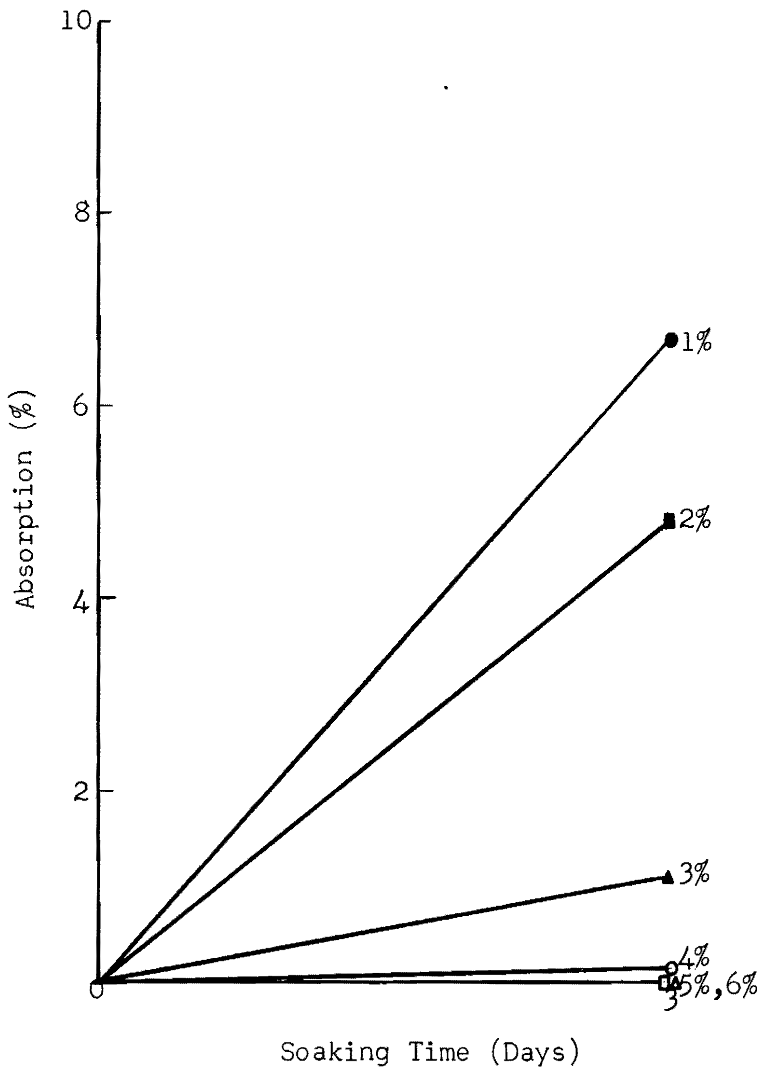

Figure 24a. Effect of MC-2 Content on Absorption (6 Days of Drying Back, No Curing).

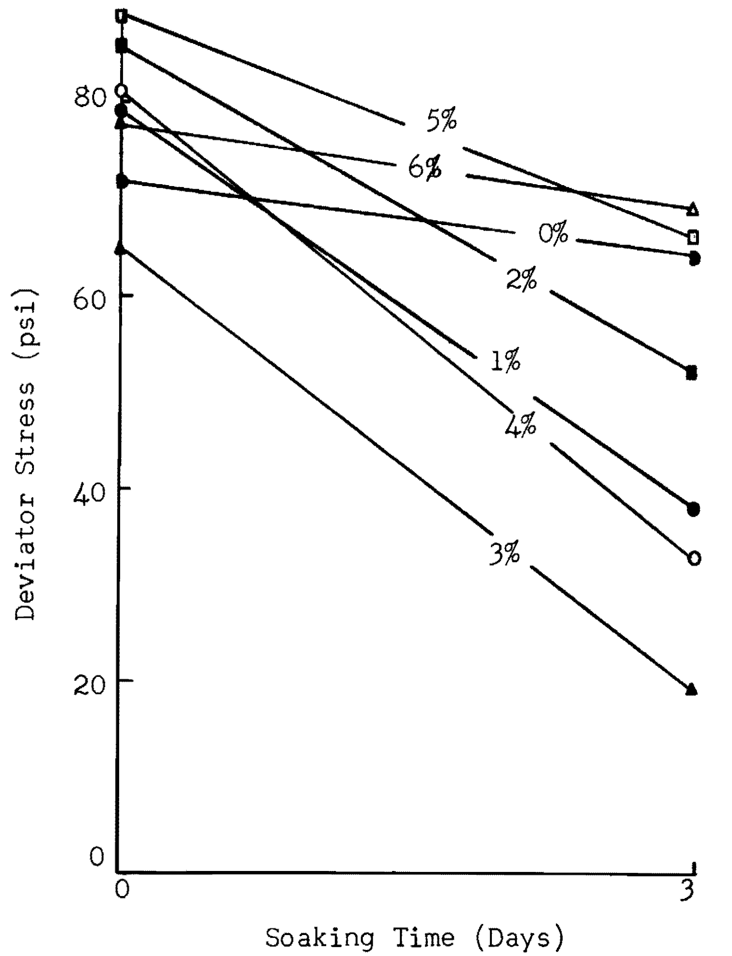

Figure 25. Effect of Soaking on Strength (No Drying-back, 3 Days of Curing).

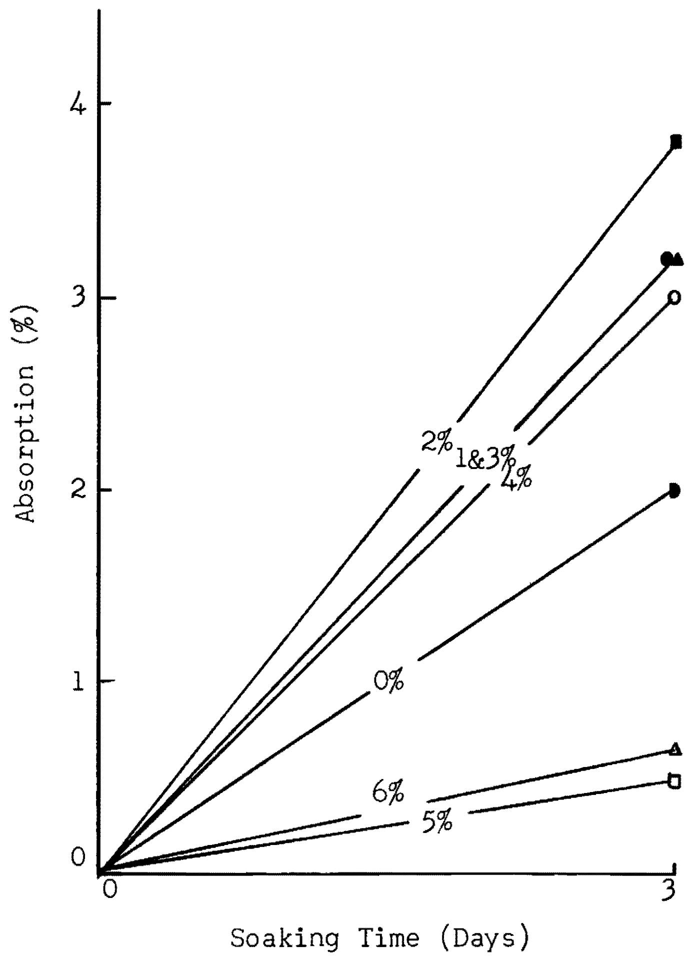

Figure 25a. Effect of MC-2 Content on Absorption (No Drying-back, 3 Days of Curing).

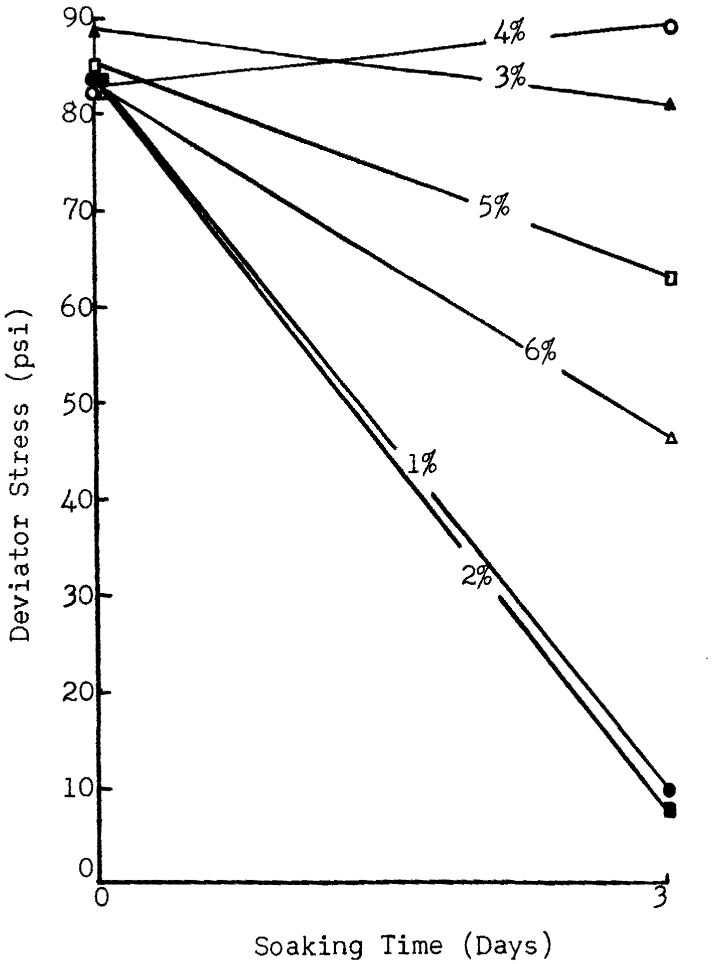

Figure 26. Effect of Soaking on Strength (3 Days of Drying-back, 3 Days of Curing).

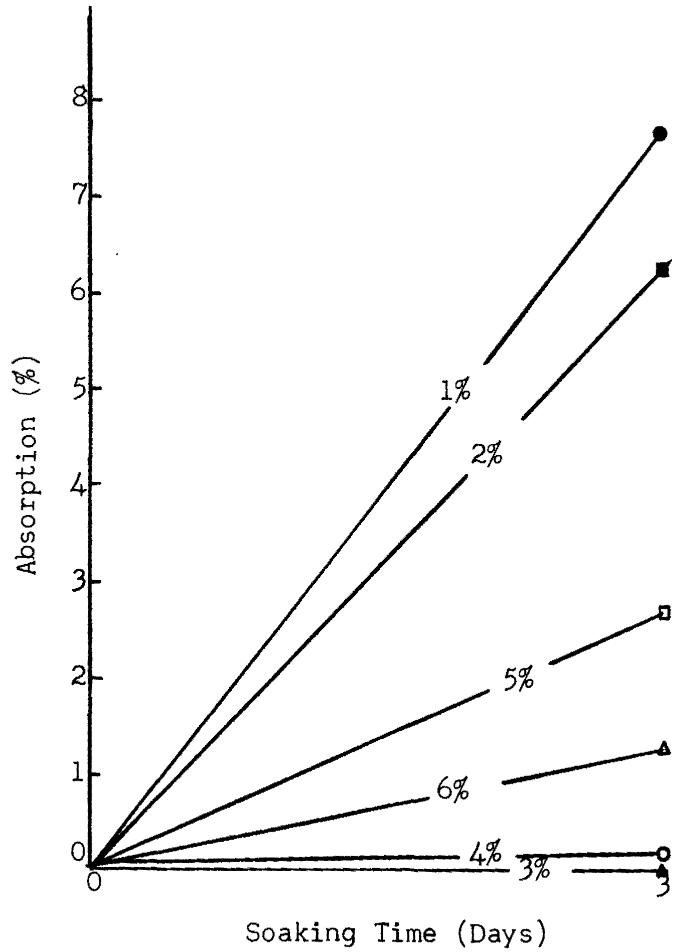

Figure 26a. Effect of MC-2 Content on Absorption (3 Days of Drying-back, 3 Days of Curing).

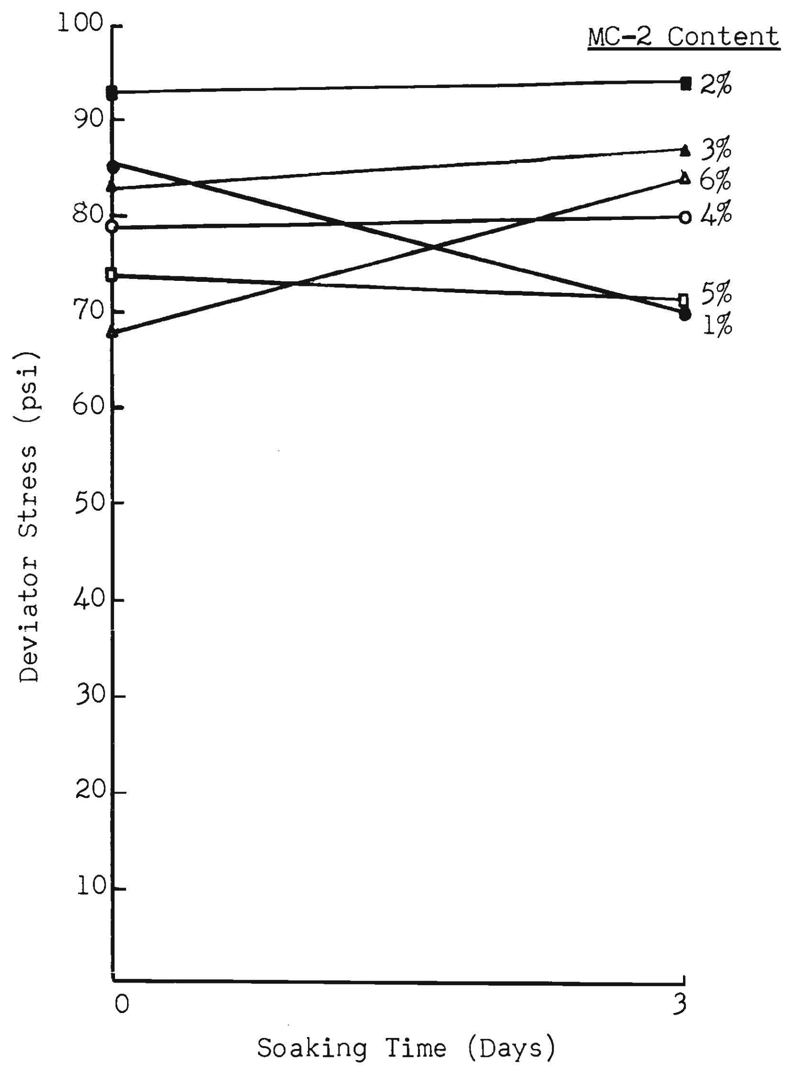

Figure 27. Effect of Soaking on Strength (6 Days of Drying-back, 3 Days of Curing).

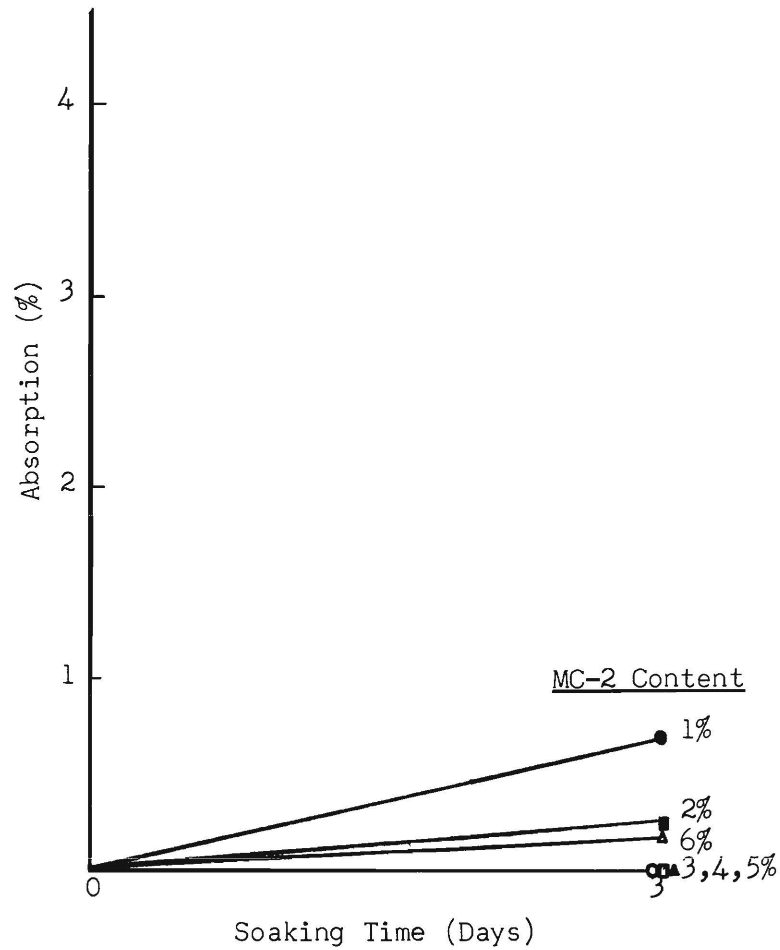

Figure 27a. Effect of MC-2 Content on Absorption (6 Days of Drying-back, 3 Days of Curing).

So far, the effects of drying back before compacting, and of soaking after compacting or curing, have been discussed. The final variable which needs to be evaluated is the curing period (after compaction). The effect of this factor is illustrated in the graphs of Figures 28 through 33. There is one especially interesting conclusion to be drawn from this series of graphs. For the samples that were soaked (refer to Figures 29, 31, and 33), those containing 1, 2, 5, and 6 per cent MC-2 generally were helped by the curing period if they were dried back for 6 days or not at all. On the other hand, the resistance to the detrimental effects of soaking to specimens containing 3 or 4 per cent asphalt was improved only if these specimens were dried back.

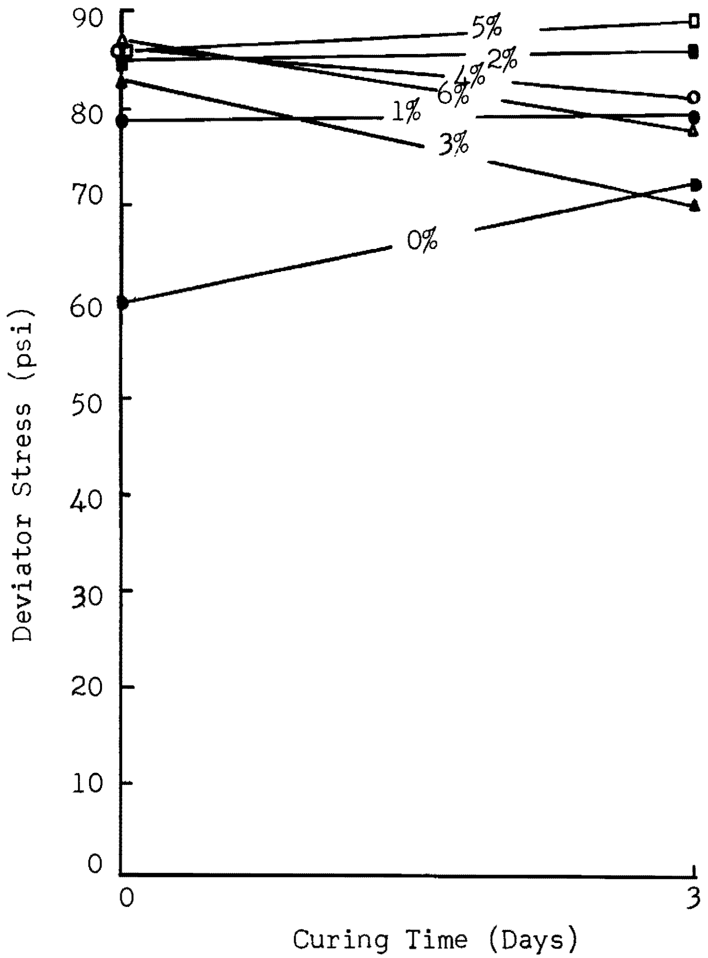

Figure 28. Effect of Curing on Strength (No Drying-back, No Soaking).

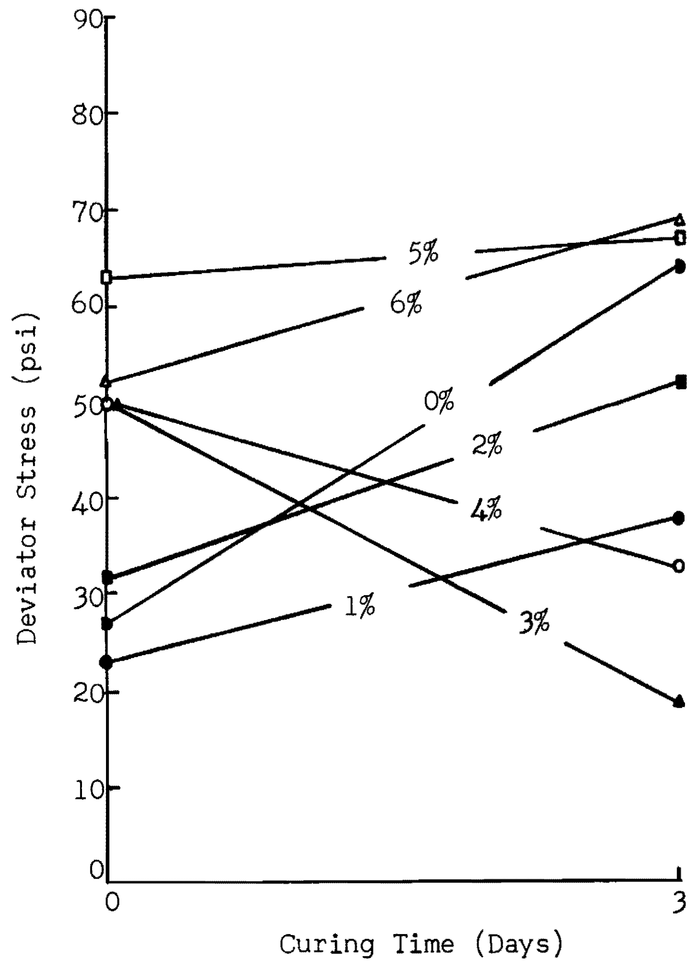

Figure 29. Effect of Curing on Strength (No Drying-back, 3 Days of Soaking).

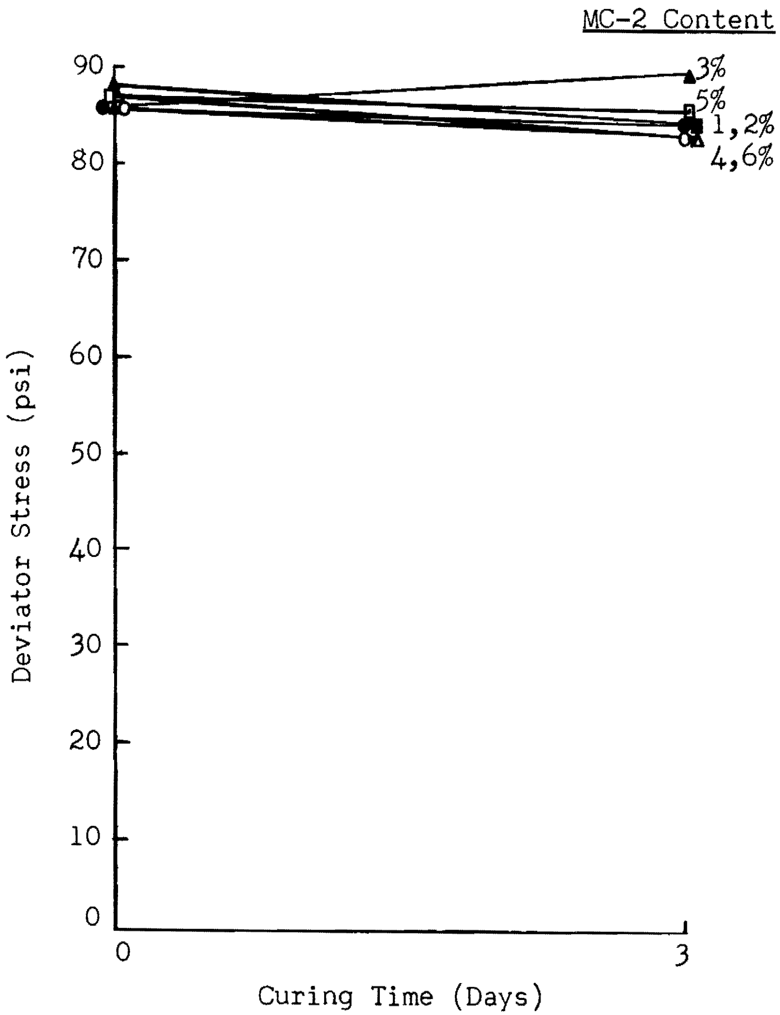

Figure 30. Effect of Curing on Strength (3 Days of Drying-back, No Curing).

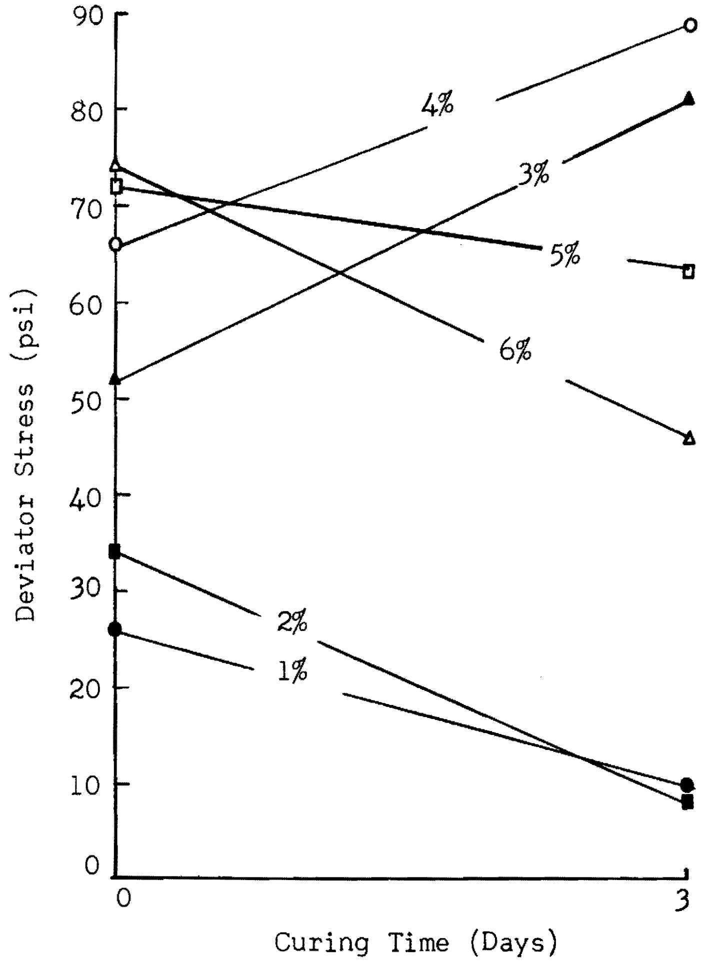

Figure 31. Effect of Curing on Strength, (3 Days of Drying-back, 3 Days of Soaking).

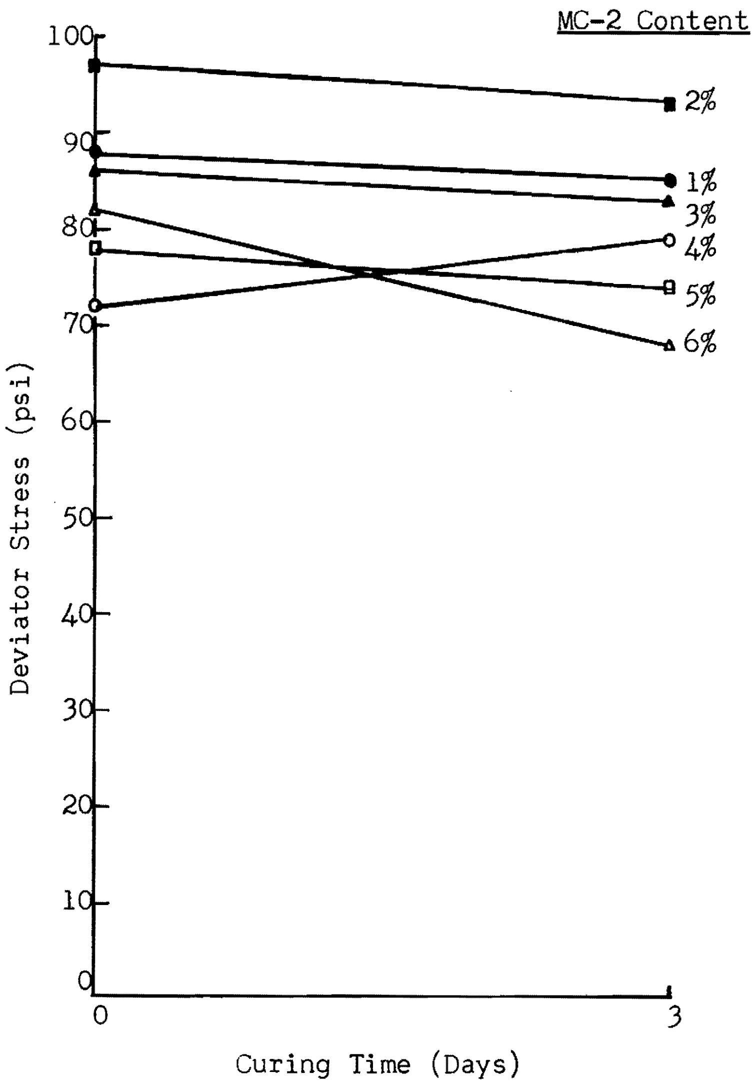

Figure 32. Effect of Curing on Strength (6 Days of Drying-back, No Soaking).

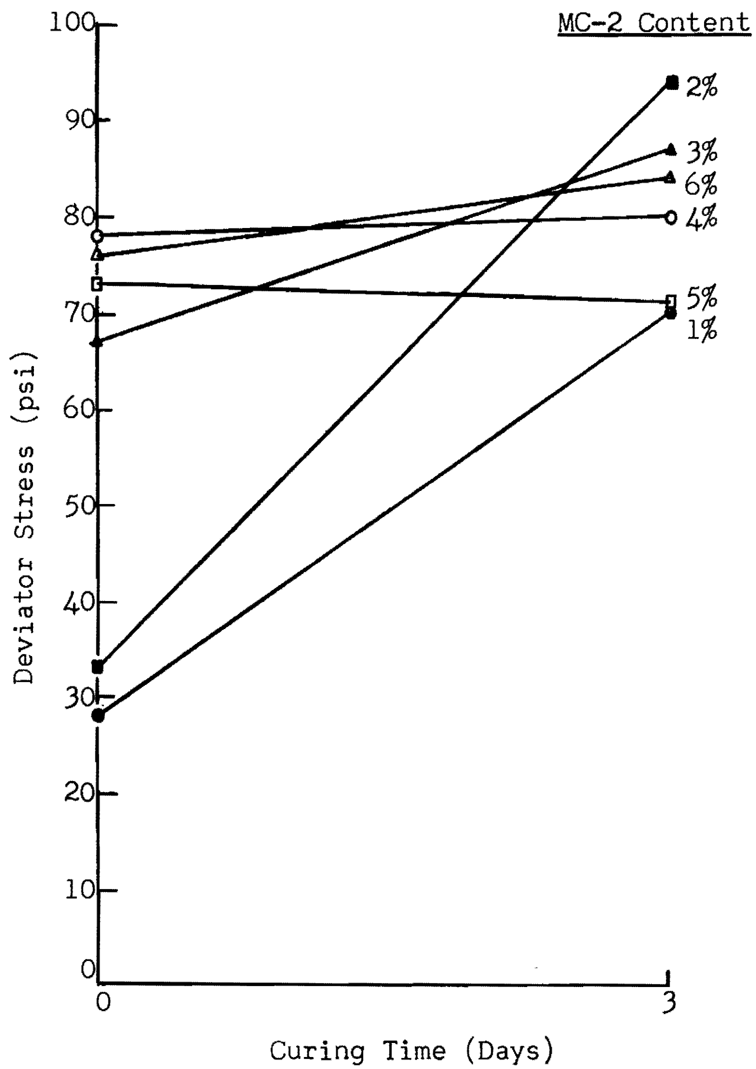

Figure 33. Effect of Curing on Strength (6 Days of Drying-back, 3 Days of Soaking).

CHAPTER VI

CONCLUSIONS AND RECOMMENDATIONS

An evaluation of the data obtained in this study leads to the following conclusions:

1. Drying mixtures of Soil XI and MC-2, up to 6 days, before compaction resulted in higher dry densities than obtained by compacting identical mixtures immediately following mixing.
2. The temperature of the MC-4 at the time it was introduced to Soil XI had little influence on the densities of the compacted mixtures.
3. For each asphalt content there is a unique combination of drying-back before compaction and curing after compaction which yields the most stable sample under a certain condition of soaking.

It is recommended that:

1. a more extensive study, utilizing known temperatures and humidities, be made on the drying characteristics of soil-asphalt mixes.
2. a similar testing program using drying-back times and curing times between 0 and 3 days be conducted.
3. an investigation be made for the purpose of comparing field methods with laboratory methods for soil-asphalt mixes; emphasis should be placed on degree of mixing; degree of drying and curing; and type of compaction.
4. a variety of soils, ranging from sand to cohesive material, be subjected to a testing program similar to the one described herein; with different types of stabilizers being used.
5. the effect of repeated soaking periods be studied.

BIBLIOGRAPHY

1. Sowers, G. B. and Sowers G. F., Introductory Soil Mechanics and Foundations, New York: The Macmillan Company, 1961, pp. 301-309.

2. Paquette, R. J. and McGee, James D., "An Investigation to Determine the Economy and Practicality of Using Various Type Soils Treated with Portland Cement or Other Admixtures for Highway Construction," Annual Report No. 1, Project No. B-136, Engineering Experiment Station, Georgia Institute of Technology, 1958.

3. Paquette, R. J. and McGee, James D., "An Investigation to Determine the Economy and Practicality of Using Various Type Soils Treated with Portland Cement or Other Admixtures for Highway Construction," Technical Report, Project No. B-136, Engineering Experiment Station, Georgia Institute of Technology, 1959.

4. Paquette, R. J. and McGee, James D., "An Investigation to Determine the Economy and Practicality of Using Various Type Soils Treated with Portland Cement or Other Admixtures for Highway Construction," Annual Report No. 2, Project No. B-136, Engineering Experiment Station, Georgia Institute of Technology, 1960.

5. Paquette, R. J. and Gale, W. Davison, "An Investigation to Determine the Economy and Practicality of Using Various Type Soils Treated with Portland Cement or Other Admixtures for Highway Construction," Annual Report No. 3, Project No. B-136, Engineering Experiment Station, Georgia Institute of Technology, 1961.

6. Paquette, R. J. and Meyersohn, Charles, "An Investigation to Determine the Economy and Practicality of Using Various Type Soils Treated with Portland Cement or Other Admixtures for Highway Construction," Annual Report No. 4, Project No. B-136, Engineering Experiment Station, Georgia Institute of Technology, 1962.

7. Katti, R. K., Davidson, D. J., and Sheeler, J. B., "Water in Cutback Asphalt Stabilization of Soil," Bulletin 241, Highway Research Board, 1960, pp. 14-48.

8. Standard Specifications for Highway Materials and Methods of Sampling and Testing, The American Association of State Highway Officials, 1958.

9. Benson, J. R. and Becker, C. J., "Exploratory Research in Bituminous Soil Stabilization," Proceedings, Association of Asphalt Paving Technologists, 1942, pp. 120-181.

10. Endersby, V. A., "Fundamental Research in Bituminous Soil Stabilization," Proceedings, Highway Research Board, 1942, pp. 442-459.

11. Herrin, Moreland, "Drying Phase of Soil-Asphalt Construction," Bulletin 204, Highway Research Board, 1958, pp. 1-13.

REFERENCES NOT CITED

1. McKesson, C. L., "Soil Stabilization with Emulsified Asphalts," Proceedings, HRB; 1935.

2. Michaels, A. S. and Puzinauskas, Vytautas, "Additives as Aids to Asphalt Stabilization of Fine-Grained Soils," HRB Bulletin 129.

3. Winterkorn, H. F. and Eckert, G. W., "Physico-Chemical Factors of Importance in Bituminous Soil Stabilization," Proceedings, AAPT; January, 1940.

4. "Soil-Asphalt Roads," Asphalt Institute Quarterly; October, 1950.

Final Report

Project No. B-136 [HPS-1(54)]

AN INVESTIGATION TO DETERMINE THE ECONOMY AND PRACTICALITY OF USING VARIOUS TYPE SOILS TREATED WITH PORTLAND CEMENT OR OTHER ADMIXTURES FOR HIGHWAY CONSTRUCTION

By

Radnor J. Paquette and James R. Fister

Contract with the State Highway Department of Georgia in Cooperation with the Bureau of Public Roads

July 1963

Engineering Experiment Station
GEORGIA INSTITUTE OF TECHNOLOGY
Atlanta, Georgia

ENGINEERING EXPERIMENT STATION
of the Georgia Institute of Technology
Atlanta, Georgia

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CONTRACT WITH THE STATE HIGHWAY DEPARTMENT
OF GEORGIA
IN COOPERATION WITH THE
BUREAU OF PUBLIC ROADS

AUGUST 1963

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Credit is also due to the following graduate students who have worked on the project and made valuable contributions which make up the final report: Mr. James D. McGee, W. Davison Gale, and Charles Meyersohn.

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CHAPTER I

INTRODUCTION

The success of any highway pavement is primarily dependent on two factors: (1) the ability of the pavement system to withstand the most critical conditions of loading imposed on it, and (2) protection of the pavement components against the elements of nature to such a degree that the desirable properties of the structure are maintained throughout its design life.

The base and subgrade courses are the most critical components of the pavement system as they must, for economy reasons, be composed mostly of local soil. Since good, natural roadbuilding materials are not in abundance in many parts of the world, it is often necessary to improve the physical properties of the available material in order to fulfill the first requirement named above. For soils which have adequate strength under normal conditions but lose strength during periods of soaking, such as are caused by high water table, it is necessary to use protective measures. The processes used to improve the strengths of natural soils or to preserve the natural strength properties of a soil come under the general heading of "stabilization".

Many different methods of stabilization have been used successfully in the past. Some of these methods are still in the development stages with economy being the biggest drawback to practical use. The most commonly used methods of soil stabilization today are: (1) mechanical stabilization, in which the gradation of the soil is altered by the blending in of other soil or crushed stone, thus producing a more compact and stable mixture; (2) cementing, in which portland cement is used to increase the cohesion; and (3) moisture resistance, in which bituminous material is mixed into the soil as a waterproofing agent in order to minimize swell and prevent loss of strength.

In Georgia, mechanical stabilization and cementing are the most widely used methods of soil stabilization.

The three methods of soil stabilization described above have been used with and without success, the success often being due to a high factor of safety. Also, there is a tendency to apply designs and construction procedures which have been used elsewhere to local conditions. Because of the infinite variety of soils and climatic conditions existing, such generalizations should not be made in the field of soil stabilization.

This research project, which began January 1, 1958, was undertaken for the purpose of investigating the economy and practicality of certain materials as stabilizers for Georgia's highway bases and subgrades.

This investigation used several methods of stabilization on twelve distinct soils by the addition of various proportions and types of admixtures. The admixtures used were Portland cement, RC-3 cutback asphalt, phosphoric acid (85% solution), lime-flyash combination, and MC-2 and MC-4 cutback asphalts. The parameters used for evaluating the affects of the different stabilizing materials were confined and unconfined compressive strength, curing methods, curing times, moisture contents and application temperature of admixtures. Upon completion of the stabilization investigation, a pilot study was made on determining how certain factors effect early cracking that commonly occur in Portland cement-treated bases.

All of the soils used in this project were secured throughout the state of Georgia and were designated as Soils I, II, III, IV, V, V-A, VI, VII, VIII, IX, X, XI, A, B, C, and D. The properties and classifications of these soils are shown in Table I.

The soils designated by Roman numerals were used in the stabilization investigation while those designated by alphabetic letters were used in the

Table I. Description of Soils

Soil No.	I	II	III	IV	V	VI	VII	VIII	IX
Location by County	Carroll	Effingham	Camden	Fulton	Fulton	Gordon	Clayton	Putnam	Putnam
Textural Analysis % retained by weight									
Sieve No. 10	3	0	0	3	2	24	1	2	1
Sieve No. 40	14	54	2	19	24	49	30	7	6
Sieve No. 60	37	68	7	28	36	52	47	11	14
Sieve No. 100	44	74	53	37	46	54	56	15	24
Sieve No. 200	62	83	92	46	55	56	61	17	30
Silt Sizes, %	21	2	3	22	24	7	23	20	31
Clay Sizes, %	6	11	--	27	14	31	16	60	33
Specific Gravity	2.67	2.63	2.69	2.70	2.69	2.67	2.59	2.67	2.63
Liquid Limit	13	14	--	29	37	20	24	64	47
Plastic Limit	--	--	--	23	--	--	14	48	44
Plasticity Index	NP	NP	NP	6	NP	NP	10	16	3
AASHO Classification	A-2-4-(0)	A-2-4(0)	A-3-(0)	A-4-(4)	A-4-(2)	A-1-a(0)+	A-4-(1)	A-7-5(15)	A-5(8)
Ga. Highway Classification	C-1 Topsoil	A-1 Topsoil	A-1 Subgrade	I-B Embankment	II-A Embankment	Class A+ Chert	B-11 Subgrade	III-B Embankment	II-A Embankment

+Note: Soil VI analysis represents only the minus 4 material but the classification includes the plus 4 material.

Table I. Description of Soils (Cont.)

Soil No.	V-A	X	XI	A	B	C	D
Location by County			Crisp	Bartow	Bartow	Fulton	Douglas
Textural Analysis % retained by weight							
Sieve No. 10	1	0	1	0	15	1	4
Sieve No. 40	16	9	29	12	39	17	37
Sieve No. 60	23	41	53	28	48	26	48
Sieve No. 100	29	79	72	55	57	38	57
Sieve No. 200	35	99	84	63	59	48	63
Silt Sizes, %	51	1	11	24	14	25	16
Clay Sizes, %	14	0	5	6	24	16	15
Specific Gravity	2.77	2.62	2.59	2.71	2.76	2.71	2.67
Liquid Limit	42	--	--	20.7	23	27	14.7
Plastic Limit	--	--	--	18.0	14.2	11	13.5
Plasticity Index	NP	NP	NP	2.7	8.8	16	1.2
AASHO Classification	A-5(6)	A-2(0)	A-2-4(0)	A-4(0)	A-4(1)	A-6(6)	A-4(0)
Ga. Highway Classification				A-1 Subgrade	C Subgrade	$11B_1$ Embankment	1-A Embankment

cracking study.

This research was performed in six different phases. The first phase was initiated in order to evaluate four different Georgia soils stabilized with Portland cement. This included a study of the susceptibility of these soils to cement treatment, measurement of the common physical properties of the various soil-cement combinations, and a determination of design requirements for soils which were to be used in future highway construction in Georgia.

The second phase was concerned with comparing the effectiveness of various stabilizers on five different soils located in the state of Georgia. The admixtures used were Portland cement, RC-3 cutback asphalt, phosphoric acid (85% solution), and a lime-flyash combination. Five soils were used in this work and are referred to as Soils I, II, III, IV, and V. The parameters used for evaluating the effects of the different stabilizing materials were confined and unconfined compressive strengths. A study of the influence of Portland cement on cohesion and on the angle of internal friction was also made.

During the third phase of this research program four more soils, designated as VI, VII, VIII, and IX were subjected to the same type testing that had already been done for Soils I through V. In addition, the effects of molding moisture content on the 28-day compressive strength was evaluated for Soils I, II, III, IV, V, VI, and VII. Also, a determination was made of the effect of various curing methods and of different moisture conditions, such as capillary soaking, on the compressive strength of soil with or without cement added.

The fourth phase of this long-range soil stabilization project was concerned with the use of RC-3 cutback asphalt as a stabilizing agent. Soils I through IX, with the exception of Soils V and V-A, were combined with various percentages of RC-3 and molded at maximum densities and optimum moisture contents. The samples compacted at moisture contents less than the optimum were mixed at the optimum moisture content and dried back before compaction.

The fifth phase of the research was composed of two distinct parts. The first part was concerned with stabilizing soil with various combinations of stone screenings (from rock-crushing operations) and Portland cement. Five different soils found in Georgia were utilized in this study. The percentages of stone screenings used were 0, 25, 50, and 75 and the percentages of Portland cement added were 2, 4, 8, and 12. Moisture-density relationships were determined for each soil alone and for each soil combined with various combinations of stone screenings and Portland cement.

The second part of this phase was concerned with studying some of the problems involved in utilizing medium curing cutback asphalt, Grade 2, as a stabilizing agent. Because of the many variables involved;[1+] a series of pilot studies were conducted before a regular testing program was organized. Some of the variables considered were asphalt temperature at the time of introduction to the soil, mixing time, and drying time before compaction. Also, during this period, an extensive testing program was systematized utilizing the results of the pilot studies.

The sixth and the final phase of the research was a study to determine how certain factors affect cracking that commonly occurs in cement treated bases. Emphasis was placed on cracking which occurs at an early age i.e., during the curing period. The factors used were derived from assumptions based on either theoretical and/or empirical knowledge.

The first assumption was that most cracking in soil-cement mixtures is attributed to the clay that is present in the soil. Other assumptions were

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+ Refer to Bibliography for numbers.

that moisture content, cement content, and temperature differential are factors in cracking. Tests were developed in order to test each assumption.

CHAPTER II

SOIL-CEMENT STABILIZATION

During the past twenty-five years the use of Portland cement for stabilizing soils for base course construction has become widespread in North America and in a number of other countries. The method of combining quantities of portland cement and local soils or aggregates, referred to as soil-cement, has been found both economical and practical under practically all climatic conditions. Its performance record has been outstanding.

This rapidly increasing use of soil-cement has resulted in the need for more engineering data on its strength and elastic properties and its ability to spread and carry traffic loads. This type information will permit engineers to take full advantage of the structural properties of soil-cement.

The overall objective of using cement as an additive in this research was to determine the susceptibility of certain soils to cement treatment, measurement of the common physical properties of the various soil-cement combinations, and a determination of design requirements for soils which were to be used in future highway construction in Georgia.

Physical Properties of Soil-Cement

In establishing laboratory procedures for the testing of any material which is to be used in field construction, the ideal situation is to simulate the field conditions in the laboratory. As this is not always possible, the laboratory procedures should include, as a primary objective, various tests which can measure the many factors affecting the material. The laboratory procedure should measure these factors in such a manner that an evaluation can be made and the laboratory results correlated with actual field conditions.

Evaluation of the load supporting characteristics of a highway base course and subgrade entails the determination of stresses within the various layers of the pavement system. Therefore, the laboratory tests should include evaluation of properties of the material to carry these stresses. This can best be accomplished by the triaxial compression test. Normal and shearing stresses are measured in the triaxial test and the effects of lateral or confining stresses can be evaluated. Also the test permits determination of the fundamental strength characteristics of a material, cohesion and angle of internal friction. For these reasons, the triaxial compression test was used along with the unconfined compression test to determine physical properties of soil-cement.

Materials Used and Test Methods

Soils Used

There was a total of nine soils used in these tests, Soil I, II, III, IV, V, VI, VII, VIII and IX. These soils were subjected to 0, 2, 4, 6, 8, 10 and 12 per cent cement contents and were cured for a period of 7 and 28 days.

These 9 soils represent the typical roadbuilding soils found in Georgia. A description of these soils is given in Table 1.

Cement

Type I portland cement purchased on the open market was used. The cement percentage was computed from the dry weight of the soil.

Preparation and Mixing of Soils

The soils were air-dried to a uniform moisture content and sieved through a No. 4 sieve with all material retained on that sieve being discarded. Standard procedures were used for the grain size analysis and Atterberg limit tests.

Mixing was done in a mechanical mixer at the lowest speed (144 RPM) using a flat beater. The air-dry soil and cement were first mixed one minute, then the water for the desired moisture content added and mixing continued for nine minutes. Each batch consisted of soil or soil-cement mixture for four specimens.

Moisture-Density Test

Moisture-density relations were performed in accordance with AASHO T134-57 modified in most instances by using a separate batch of soil for each test point. The test was performed on the raw-soil and at each cement content investigated in the later strength tests.

Molding Test Specimens

The test specimens were 2.8 inches in diameter and 5.6 inches high. Since the soils studied were predominantly fine-grained materials, the static method of compaction was used. Pistons were used in each end of the 2.8 inch diameter mold to compact the sample. To insure uniform density both pistons were allowed to move which effected a force from each end of the sample. For better compaction of the center portion, a 5/8 inch rod was used to compact the soil as it was placed in the mold in two layers. The amount of soil or soil-cement mixture placed in the mold was the weight to give maximum density with a 5.6 inch height. The two pistons were then forced together by a 120, 000 pound constant-stress laboratory testing machine until a dial gage indicated the proper 5.6 inch height. The molding equipment is shown in Figures 1 and 2. For each test, 4 samples were molded for unconfined compression at 7 and 28 days and 4 samples for triaxial testing at age 7 and 28 days.

Curing

Considerable thought and study was devoted to the development of a curing method that would best simulate field curing conditions. In this

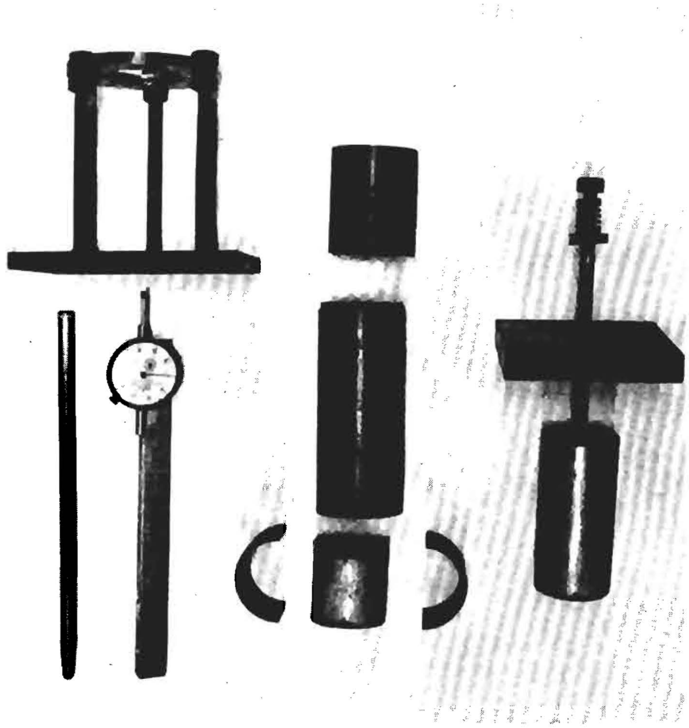

Figure 1. Molding Equipment.

Figure 2. Molding Soil-Specimen.

respect, experience has indicated that the moisture content of a compacted highway base will undergo very little change under normal curing conditions. Unless the roadway is inundated or subjected to extremely wet or dry conditions, a properly constructed base should remain very near its compacted moisture content. Laboratory studies made as part of this research program show that generally the greatest strength is obtained by curing the specimens with only the molding moisture available and without exposure to excessive moisture conditions. As a result, the laboratory specimens in this test program were cured in a condition of no moisture change. This was accomplished by sealing the specimens in polyethylene plastic bags and storing at 70°F to prevent any variation from daily fluctuations in temperature. This process is shown in Fig. 3.

Triaxial Compression Test

After 7 and 28 days curing, triaxial compressive strength tests were performed using a lateral confining pressure of 20 psi. Lateral pressure was applied by compressed air. Tests at zero lateral pressure (unconfined compression) were also included. A constant-strain screw type testing machine was used with a rate of loading of 0.05 inch per minute. Fig. 4 shows this testing equipment.

Test Results

Testing of the soils and soil-cement mixtures involved determining maximum density and optimum moisture of each mix and determination of the unconfined and confined compressive strength.

Moisture-Density Relations and Compressing Strength

Table II shows the values of maximum dry density and optimum moisture content for the soils and soil-cement mixtures. The effect of cement on these values varied with the soil gradation. Tabular results of the average

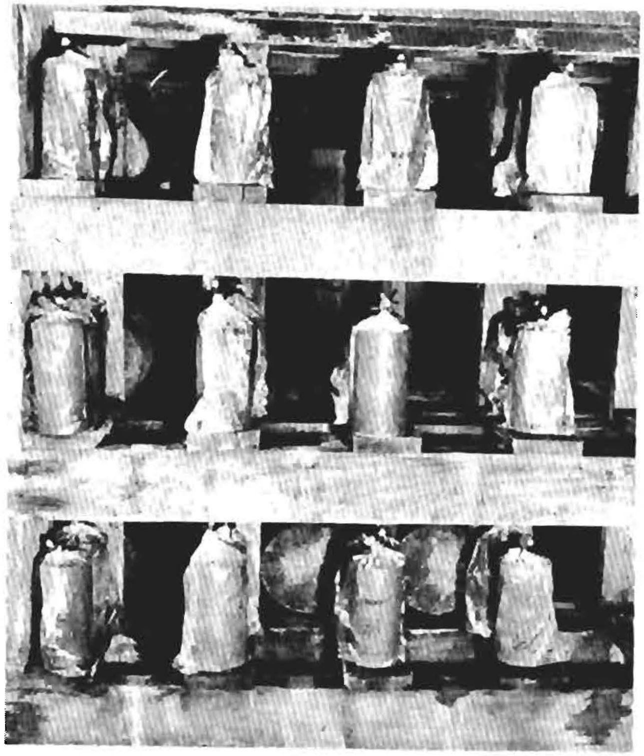

Figure 3. Curing Molded Samples.

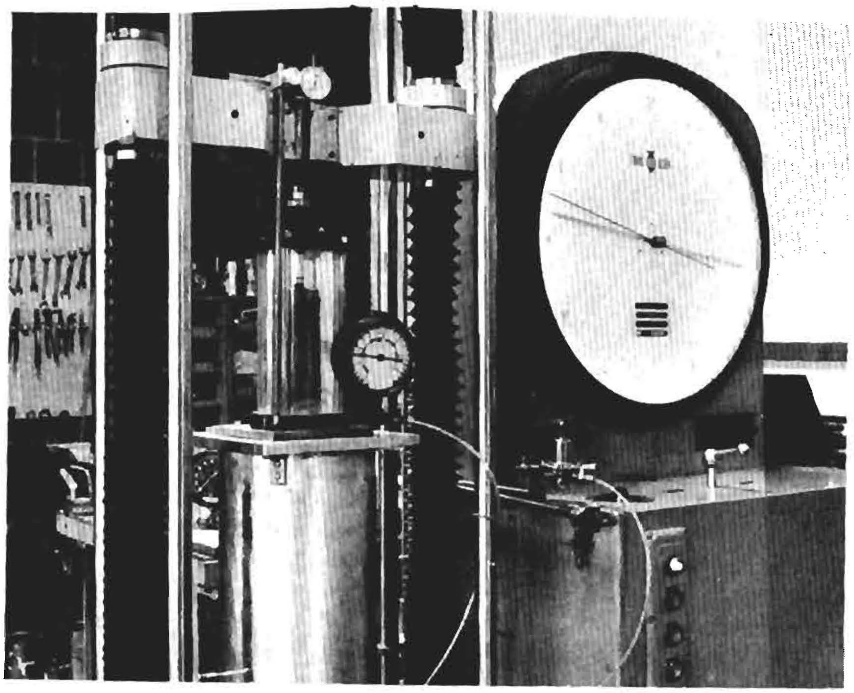

Figure 4. Constant-Strain Triaxial Equipment.

Table II. Maximum Dry Density and Optimum Moisture

Cement %	Soil I Max. Dry Density lb/ft^3	Soil I Optimum Moisture %	Soil II Max. Dry Density lb/ft^3	Soil II Optimum Moisture %	Soil III Max. Dry Density lb/ft^3	Soil III Optimum Moisture %	Soil IV Max. Dry Density lb/ft^3	Soil IV Optimum Moisture %	Soil V Max. Dry Density lb/ft^3	Soil V Optimum Moisture %
0	121.0	9.0	119.1	10.3	101.0	9.5	114.2	14.6	111.2	16.5
2	122.9	10.0	120.1	11.0	102.1	9.8	112.4	15.5	111.9	16.4
4	123.0	10.5	121.9	11.0	104.3	10.0	111.6	16.6	111.9	16.7
6	123.1	11.0	122.7	10.2	106.5	10.8	111.8	16.8	111.7	17.0
8	123.9	10.8	123.1	10.6	109.0	11.7	112.3	15.8	111.2	17.3
10	123.7	10.4	123.3	10.6	110.0	11.4	112.2	15.9	111.8	16.8
12	124.9	10.5	123.9	10.1	111.1	11.2	114.2	15.1	111.2	17.3

Table II. Maximum Dry Density and Optimum Moisture (Cont.)

Cement %	Soil VI Max. Dry Density lb/ft^3	Soil VI Optimum Moisture %	Soil VII Max. Dry Density lb/ft^3	Soil VII Optimum Moisture %	Soil VIII Max. Dry Density lb/ft^3	Soil VIII Optimum Moisture %	Soil IX Max. Dry Density lb/ft^3	Soil IX Optimum Moisture %
0	110.2	14.7	116.1	14.0	88.7	30.9	100.4	22.4
2	110.5	14.9	114.8	13.9	90.3	30.7	101.9	22.0
4	110.6	15.1	115.2	14.2	90.4	30.3	101.0	21.7
6	111.8	14.8	115.0	14.2				
8	110.6	15.3	116.1	13.8	90.0	30.9	101.5	21.5
10	110.5	15.5	114.8	13.8				
12	111.9	14.3	117.3	13.4	90.6	30.5	103.3	20.5

strength test are given in Table III.

For Soil I, the addition of Portland cement produced an increase in maximum density with increasing amounts of cement while the optimum moisture increased slightly with the addition of cement then remained nearly constant as the cement percentage increased. The compressive strength increased with the addition of portland cement with small gains in strength with low percentages of cement, and greater increases in strength with the higher percentages. The triaxial test curves and unconfined test curves had approximately the same shape with the triaxial test curves showing greater improvements at the lower percentages of cement. The 28 day test curves showed approximately a constant increase in strength over the 7 day test curves from 6% to 12% cement.

Soil II showed that increasing percentages of cement caused increasing density with little change in optimum moisture. The addition of portland cement produced approximately a linear increase in strength with increasing percentages of cement. There was also greater increases in the 28 day strengths over the 7 day strengths with increasing amounts of cement.

Portland cement caused a nearly linear increase in density with increase in percentages of cement for Soil III. Optimum moisture also increased but to a lesser amount at the higher cement contents. The addition of portland cement had negligible effect on strength up to 6 per cent. The addition of more than 6 per cent greatly increased the strength with a rapid rise in the strength curves up through 12 per cent. The increase in strength of the 28 day tests also was greater at the higher cement contents.

Portland cement added to Soil IV caused a slight reduction in density, the reduction becoming less at the higher percentages. Optimum moisture increased slightly with the intermediate percentages of cement with practically the same moisture content at higher percentages as with the original soil.

Table III. Average Results of Strength Tests
(Axial Stress, σ , psi)

Cement	Soil I				Soil II				Soil III				Soil IV				Soil V			
	7 Day		28 Day		7 Day		28 Day		7 Day		28 Day		7 Day		28 Day		7 Day		28 Day	
%	0+	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20
0	15	81	7	81	15	85	13	84	0	0	0	59	33	58	38	59	23	38	25	45
2	20	114	23	139	95	189	144	228	0	61	3	58	104	142	109	167	73	123	81	133
4	50	163	81	239	217	317	220	346	3	68	3	62	249	291	274	347	188	245	246	290
6	98	254	148	261	378	439	354	488	7	89	5	68	272	344	369	465	209	282	291	352
8	247	354	274	516	475	565	667	728	49	163	128	216	289	367	376	482	268	339	341	417
10	431	580	557	693	618	717	880	945	122	240	222	329	349	466	458	568	296	382	394	457
12	572	726	712	883	769	864	1035	1114	262	324	389	484	457	469	550	640	283	367	413	504

+Note: The 0, and 20 indicate confining pressure in psi in the triaxial test.

Table III. Average Results of Strength Tests (Cont.)
(Axial Stress, σ , psi)

Cement %	Soil VI				Soil VII				Soil VIII				Soil IX			
	7 Day 0+	7 Day 20	28 Day 0	28 Day 20	7 Day 0	7 Day 20	28 Day 0	28 Day 20	7 Day 0	7 Day 20	28 Day 0	28 Day 20	7 Day 0	7 Day 20	28 Day 0	28 Day 20
0	37	109	33	95	42	88	61	98	47	76	38	76	38	76	38	76
2	153	262	181	272	176	209	208	245	66	107	56	106	55	116	62	116
4	243	354	284	380	440	464	548	630	96	168	102	157	178	233	194	257
6	321	467	434	546	575	616	680	734	--	--	--	--	--	--	--	--
8	420	540	551	659	754	767	793	810	247	289	242	312	289	357	348	431
10	562	675	776	842	677	712	946	1010	--	--	--	--	--	--	--	--
12	738	782	874	963	957	971	983	1038	281	361	393	462	357	443	502	591

+Note: The 0, and 20 indicate confining pressure in psi in the triaxial test.

The addition of cement caused a marked increase in strength even with the 2 per cent addition. This increase in strength was approximately linear with the 28 day strength increasing over the 7 day strength at higher percentages.

For Soil V, the addition of portland cement had no effect on density or moisture. Portland cement was a very effective stabilizer with approximately 100 per cent increase in strength at 12 per cent. The rate of increase was greatest up to 8 per cent. A steady increase in 28 day strength over the 7 day strength was noted with increasing amounts of cement.

Little change was noted in Soils VI, VII, VIII and IX with the addition of portland cement. Soils VI and VII, both coarse-grained as compared to Soils VIII and IX, did not have a substantially higher density with the addition of cement as had been found previously in the coarser-grained soils. Soils VIII and IX both had slight density increases with the addition of cement.

Soil VI, the chert, showed high strength gains with the addition of increasing percentages of cement. This strength is nearly linear particularly with the 28 day curing period.

Soil VII, the sandy clay, had a large strength gain up through 12 per cent cement. Results with this soil were somewhat erratic but the general trend is near a linear relationship.

Soil VIII, the heavy clay, showed an increase in strength with the addition of cement. The greatest benefit occurs at the higher percentages for the 28 day tests.

Soil IX had a near linear increase in strength with the addition of portland cement. The variation in strength in the confined and unconfined results was approximately the same at 7 and 28 days with a slightly greater benefit derived from the confinement at the highest cement content.

Compressive Strength Versus Molding Moisture Content

Molding moisture content on 28 day compressive strengths was evaluated for Soil Nos. I through VII. These soils and soil-cement mixtures were molded at moisture contents 3 per cent lower and 3 per cent higher than optimum moisture with the dry density held constant at the maximum value obtained by the standard AASHO moisture-density test. The variation in strength were measured by unconfined and confined tests. Results are tabulated in Table IV. Results for Soil Nos. I, III and IV are shown in Figures 5 through 7.

Compressive Strength Versus Various Curing Methods

The effects of various curing methods were evaluated for Soils I through IX. Compressive strength was determined for each soil with four, eight and 12 per cent portland cement after curing was compared with the compressive strength of samples cured in sealed plastic bags. One batch of samples was cured under the extreme conditions of submergence or soaking. These samples were cured in the sealed bags for five days, soaked in a container of water for two days then tested in unconfined compression. The other variation in curing consisted of exposing the molded samples to capillarity moisture. These samples were placed in a container immediately after molding and allowed to stand on saturated porous mats for seven days, then were tested in unconfined compression and with 20 psi confinement. These results are shown in Table V.

Cohesion and Angle of Internal Friction

Soils I through V, which were subjected to the 50 psi confining pressure, showed a definite increase in cohesion and angle of internal friction with the addition of portland cement. The optimum percentage of cement for angle of internal friction was around six per cent. Increasing the cement

Table IV. Average Results of 28 Day Strength Tests at Various Molding Moisture Contents (Axial Stress, σ_1, psi)

Cement %	Molding Moisture Content	Soil I 0	Soil I 20	Soil II 0	Soil II 20	Soil III 0	Soil III 20	Soil IV 0	Soil IV 20	Soil V 0	Soil V 20	Soil VI 0	Soil VI 20	Soil VII 0	Soil VII 20
0	Optimum	7	101	13	104	0	79	38	79	25	65	35	95	61	98
4	3% Low	166	330	201	315	4	73	144	256	176	262	300	380	534	611
	Optimum	81	259	220	366	3	82	274	367	246	310	284	380	548	630
	3% High	139	258	285	402	3	83	231	270	206	314	165	295	471	546
8	3% Low	576	725	551	678	261	393	273	386	337	420	434	563	711	767
	Optimum	274	536	667	748	128	236	376	502	341	437	551	659	771	792
	3% High	587	692	639	684	151	307	484	483	414	510	425	497	493	600
12	3% Low	827	927	850	925	569	660	447	570	367	436	625	752	801	935
	Optimum	712	903	1035	1134	405	525	550	660	413	524	874	963	1085	1125
	3% High	1157	1191	1178	1179	415	634	635	789	568	599	803	909	987	1020

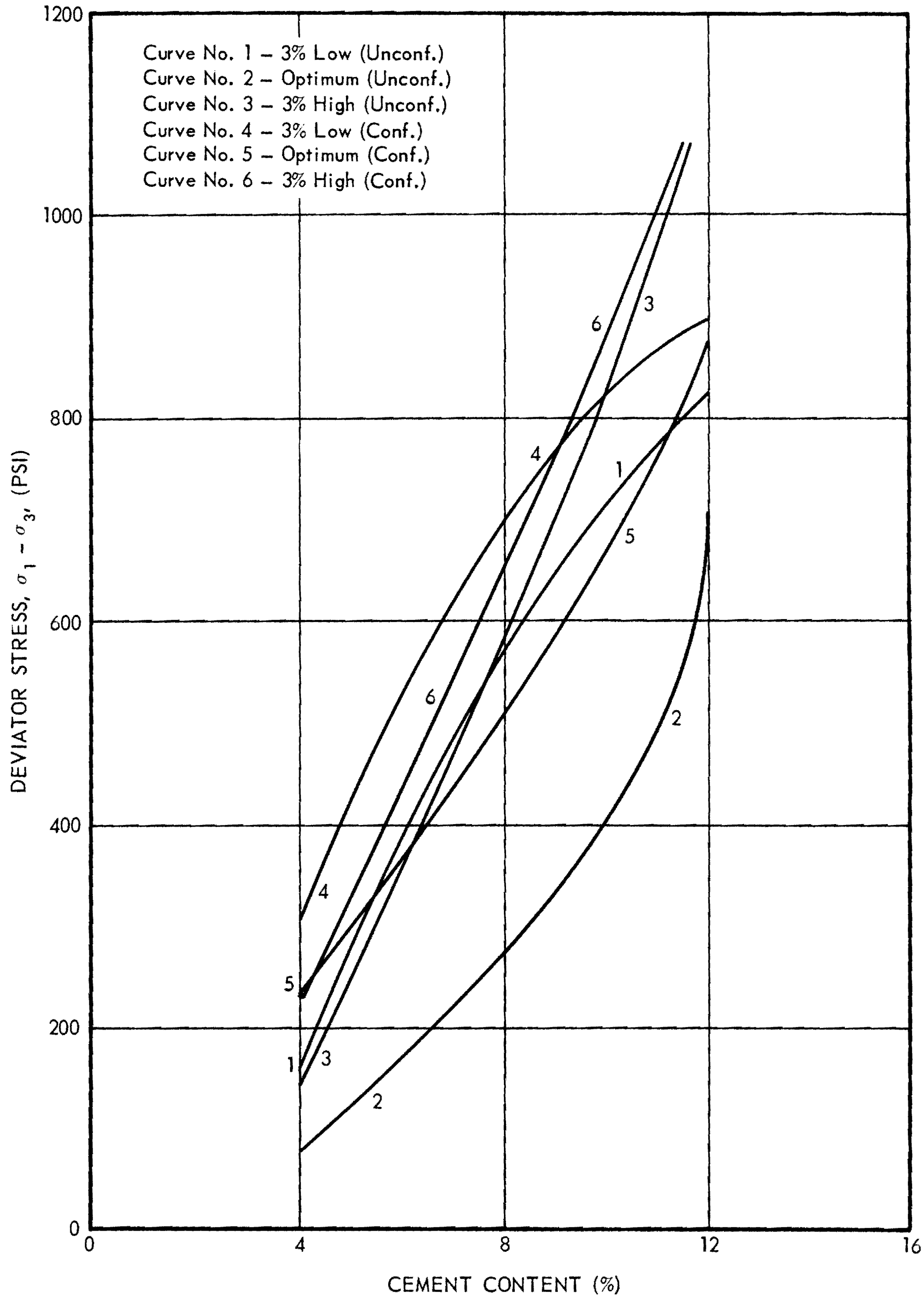

Figure 5. Deviator Stress Vs. Cement Content with Various Molding Moisture Contents for Soil I.

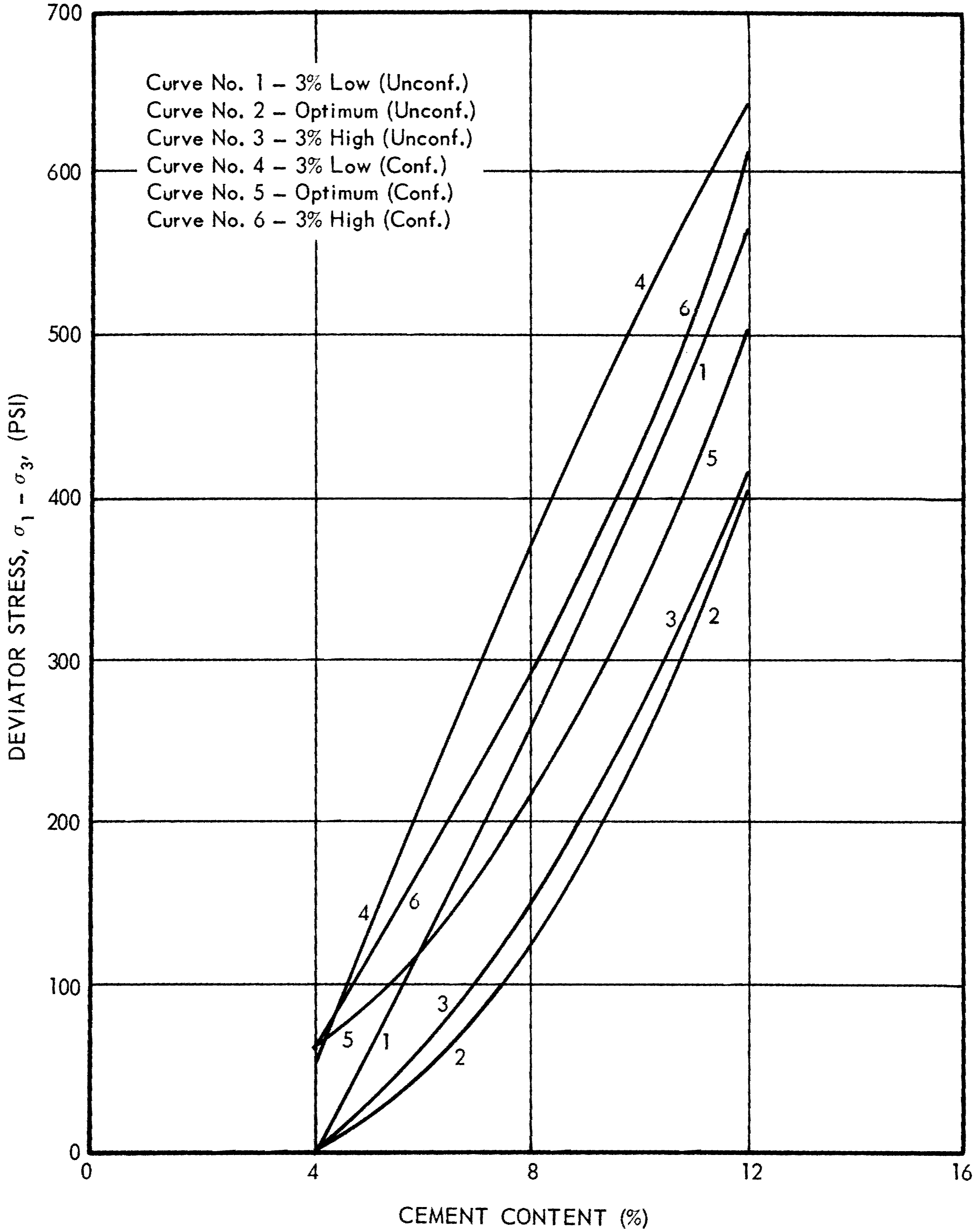

Figure 6. Deviator Stress Vs. Cement Content with Various Molding Moisture Contents for Soil III.

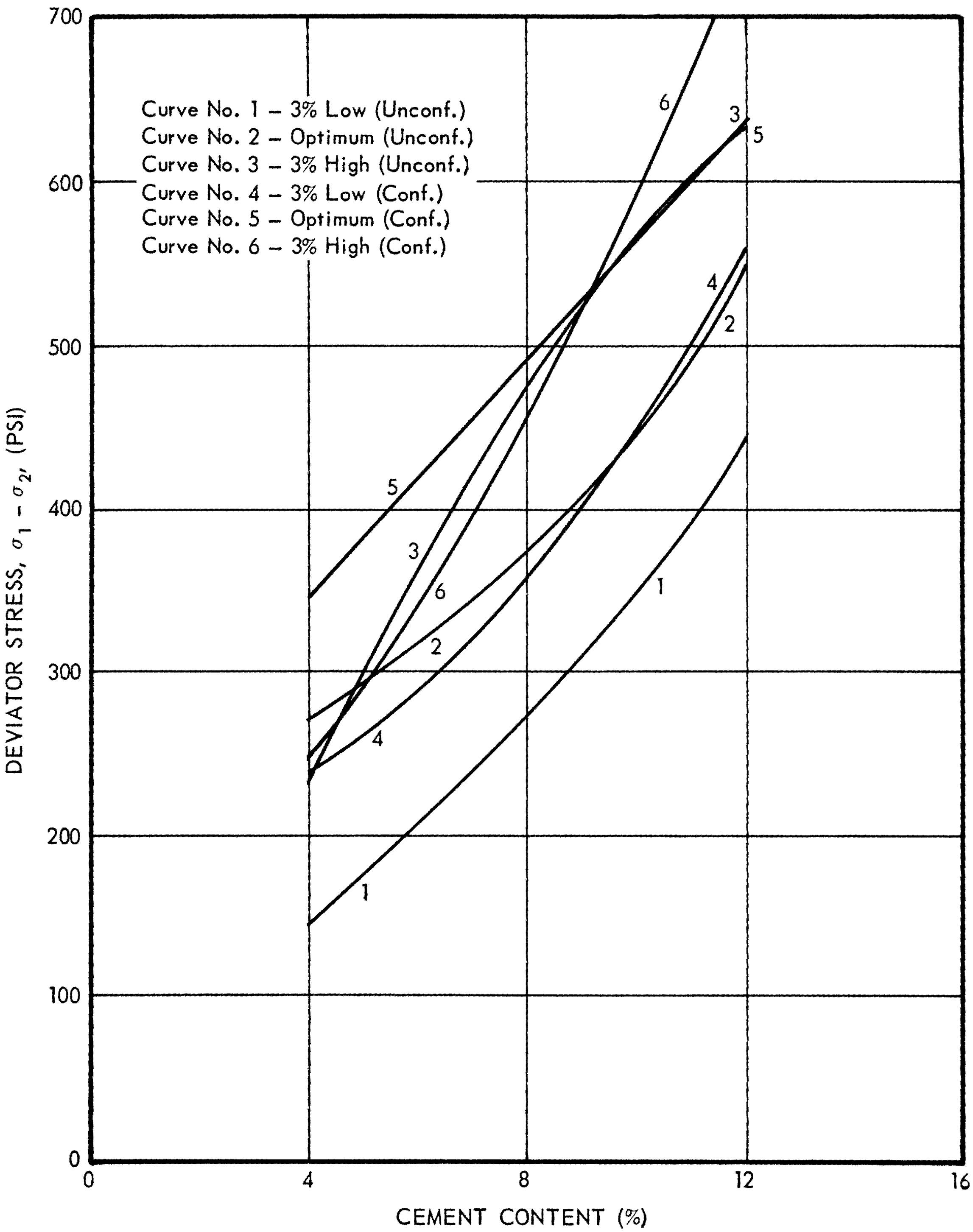

Figure 7. Deviator Stress Vs. Cement Content with Various Molding Moisture Contents for Soil IV.

Table V. Average Results of Seven Day Strength Tests After Various Curing Methods (Axial Stress, σ_1 , psi)

Cement %	Curing Method	Soil I		Soil II		Soil III		Soil IV		Soil V	
		0	20	0	20	0	20	0	20	0	20
0	Seal	15	101	15	105	0	--	33	78	23	58
	Soak	0	--	0	--	0	--	--	--	0	--
	Capillary	--	--	--	--	0	--	--	--	--	--
4	Seal	50	183	217	337	3	88	249	311	188	265
	Soak	35	--	161	--	0	--	118	--	147	--
	Capillary	40	164	151	269	0	--	76	144	123	192
8	Seal	247	374	475	585	142	244	289	387	268	359
	Soak	200	--	374	--	120	--	226	--	255	--
	Capillary	175	370	422	529	0	33	178	278	231	291
12	Seal	572	746	769	884	262	344	457	489	283	387
	Soak	400	--	572	--	235	--	360	--	293	--
	Capillary	435	648	746	785	235	346	298	422	277	329

Table V. Average Results of Seven Day Strength Tests
After Various Curing Methods (Cont.)
(Axial Stress, σ_1 , psi)

Cement %	Curing Method	Soil VI 0	Soil VI 20	Soil VII 0	Soil VII 20	Soil VIII 0	Soil VIII 20	Soil IX 0	Soil IX 20
	Seal	37	109	42	88	47	73	38	76
0	Soak	0	--	0	--	0	--	--	--
	Capillary	--	--	--	--	--	--	--	--
	Seal	243	354	440	479	96	153	178	225
4	Soak	212	--	293	--	0	--	105	--
	Capillary	163	319	289	332	21	106	47	107
	Seal	427	540	754	730	247	289	289	357
8	Soak	345	--	585	--	138	--	226	--
	Capillary	349	485	623	681	214	298	164	240
	Seal	738	782	957	971	281	361	357	443
12	Soak	528	--	749	--	263	--	297	--
	Capillary	450	627	900	928	349	400	405	458

content above that percentage resulted in very small increases in the angle of internal friction. The cohesion of each soil increased almost constantly with increasing cement percentages. Data from the Mohr's diagram is tabulated in Table VI.

Conclusion

It was concluded from the above tests that all of the soils tested can be effectively stabilized with portland cement. At the present time, the Georgia Highway Department is designing on a compressive strength of 300 psi in seven days using a triaxial test with 20 psi confinement. This design strength has been obtained in the test program for all nine soils with a cement content ranging from four per cent to 12 per cent.

The effects of moisture content at various cement contents were checked only for strength and these results indicate that in general the greatest strength at 28 days is obtained either at optimum moisture or slightly higher.

The results of the various methods of curing indicate that generally, the greatest strength is obtained by curing the samples without exposure to excessive moisture conditions either by soaking or capillarity. The samples tested after two days immersion had the lowest strength in most cases, but in some soils, exposure to capillary moisture immediately after molding greatly affected the strength. Both of these methods of curing, immersion for two days after five days sealed curing and seven days capillary curing are in our opinion extreme curing conditions.

Details of the procedure as adopted by the Georgia State Highway Department are given in the appendix.

Table VI. Mohr's Diagram Data

Soil No.	Cement Content	Compressive Strength (28 Day)			Cohesion, C	Angle of Internal friction, ϕ
	%	psi			psi	degrees
		0*	20	50		
I	0	7	81	179	2	33
	6	148	261	574	30	49
	9	564	774	862	110	49
	12	712	883	1113	130	51
	15	750	1043	1080	145	49
II	0	13	84	158	5	29
	6	354	488	702	70	48
	9	751	873	1004	162	43
	12	1035	1114	1271	239	41
	15	1310	1448	1589	275	45
III	0	0	59	142	0	29
	6	5	68	165	2	34
	9	210	305	413	52	38
	12	389	484	772	88	41
	15	793	884	1054	180	41
IV	0	38	59	86	19	0
	6	369	465	564	95	36
	9	415	466	569	120	30
	12	550	640	836	136	40
	15	647	744	862	153	39
V	0	25	45	73	12	0
	6	291	352	440	83	31
	9	403	492	551	115	31
	12	413	504	585	123	31
	15	454	608	672	136	31

*Note: The 0, 20, and 50 indicate confining pressure in psi in the triaxial test.

CHAPTER III

BITUMINOUS STABILIZATION

On a typical highway construction project, many widely varying soil types are likely to be encountered. Consequently, a desirable characteristic of a stabilizing process or type of admixture would be the ability to benefit the engineering properties of different soil types.

Cutback asphalt is an asphalt cement that has been liquefied by blending with petroleum diluents. Upon exposure to atmospheric conditions, the diluents evaporate leaving the asphalt cement as a residue. This base asphalt has two characteristics, both of which may be beneficial in soil stabilization. Asphalt cement acts to some degree as a cementing agent thereby introducing cohesion to granular materials and increasing the stability of the combined materials. On the other hand asphalt cement is a waterproofing agent that can render water sensitive soils stable by preventing the intrusion of moisture.

The object of this research is to evaluate the factors influencing the soil-water-cutback asphalt stabilization mechanism and to develop a laboratory procedure for the design and control of bituminous stabilized bases and subgrades.

Several representative soil types found in the state of Georgia were selected to determine the susceptibility of these soils to stabilization by the addition of a bituminous material. These different soil types were combined with cutback asphalt to determine the optimum asphalt content for each soil. In order to achieve maximum resisting characteristics from the soil-water-cutback asphalt mixture it was necessary to determine:

1) the moisture-density relationships of the materials involved

that coincide with maximum strength.

2) the effectiveness of cutback asphalt as a "lubricant" in compaction.

3) the correct mixing, curing, compaction and strength testing cycle.

4) the waterproofing properties of cutback asphalt.

The triaxial shear test was chosen as the criteria for evaluating stability soil-water-cutback-asphalt mixtures for the following reasons:

1) The desired result of a stabilization process is an increase in stability. Compressive strength as evidenced in a triaxial shear test is a measure of stability.

2) The triaxial shear test is a familiar laboratory procedure that does not require special equipment other than that normally found in a soil testing laboratory.

3) The merits of other admixtures have been judged by this test and correlation with research of this nature will be provided.

At the recommendation of the Georgia State Highway Department RC-3, MC-2 and MC-4 were selected as the cutback asphalts to be used in this investigation.

Physical Properties of Bituminous Admixtures

Full scale tests incorporated in a correlation study by Endersby[2] indicated the presence of high confining pressures imposed through paving restraint. This paving restraint, according to Endersby gave rise to higher stability values as compared to similar soil-cutback asphalt mixtures tested in the laboratory.

Prandtl[3] devised a bearing power test that introduced the effects of paving restraint as well as measuring the values of cohesion and internal friction. Comparing the results of his bearing power test with results of unconfined compressive tests Prandtl reported values of bearing power ten times that of unconfined compressive strength. The fact that full size pavement sections contribute confining pressures in excess of those imposed on laboratory samples has been evidenced in existing bases and subgrades that have performed satisfactorily despite laboratory tests indicating insufficient stability.

The preceding information indicates a need for research in stress distribution beneath various types of surface and base materials.

In all probability, the most argumentative aspect of bituminous stabilization is the relationship between moisture and cutback asphalt in the compaction characteristics of bituminous stabilized materials. Cutback asphalt is composed of asphalt cement and a gasoline or naptha diluent, the volatility of which depends on the particular grade. The volatiles present in cutback asphalt serve to some extent as a lubricating medium in compaction in much the same manner as water. Whether one per cent volatiles exhibits the same effect on the compaction characteristics of a particular soil as one per cent water is a much discussed topic with widely varying theories represented.[4,5]

The American Road Builder's Association[6] makes the following suggestions as to the physical characteristics of the soil material that is to be stabilized with cutback asphalt:

1) Per cent passing a No. 4 Sieve $\geq$ 50.
2) Per cent passing a No. 40 Sieve, 50-100.
3) Per cent passing a No. 200 Sieve $\leq$ 35.
4) Liquid limit should be less than 30%.
5) Plasticity index must be less than 10.

The effects on density and strength of compacted mixtures appear to depend on the gradation characteristics of a soil. The principal function of asphalt in a cohesive soil is to waterproof the consolidated soil mass. Findings such as these are reported in a paper prepared by Puzinauskas and Kallas.[7]

Materials Used and Test Methods

Soils Used

Soils I through XI, with the exception of Soil V-A, were used in this bituminous admixture study. A detailed description of the physical characteristics of each soil is presented in Table I.

All soil classification tests to identify these soils conform to standard recommended practices of the American Association of State Highway Officials as well as recommendations in soils testing manuals.[8]

Rapid Curing Cutback Asphalts

Rapid curing cutback asphalt (RC-3) is an 80-120 penetration asphalt cement that has been liquefied by blending with petroleum diluents of high volatility such as naptha or gasoline.

The RC-3 utilized in this research was supplied by the Savannah, Georgia Refinery of the American Oil Company. The following is a typical chemical analysis of this material:

Flash Point (Open Tag)............	95°
Saybolt Viscosity at 140°F........	438
Distillation Test:	
Distillate, percentage by volume of total distillate:	
374°F..........................	19%
437°F..........................	60
500°F..........................	76
600°F..........................	91
Residue from distillation to 680°F..........................	79%
Specific Gravity at 60°F..........	0.9759
Residue Penetration at 77°F.......	90
Ductility at 77°F................	100+
Solubility in CCl_4................	99.9%
Spot test.........................	Negative

Medium-Curing-Cutback Asphalt

The medium-curing cutback asphalt used was obtained from the Shell Oil Company refinery in Savannah, Georgia. Distillation, penetration, viscosity, and specific gravity tests were conducted periodically on this material. It was found that these properties varied very little for the asphalt used throughout the research work. Typical values of the properties determined are shown below. All asphalt used met the specifications of the American Society for Testing Materials for the medium-curing asphalts, Grade 2 and 4.

Characteristics	ASTM Test Method	Grade 2	Grade 4
Flash Point (Open Tag), °F	D 1310	230	221
Furol Viscosity at 140°F, seconds	D 88	163	
Furol Viscosity at 180°F, seconds	D 88		194
Distillation:	D 402		
Distillate (% of total distillate to 680°F)			
To 437°F		0	0
To 500°F		30	12
To 600°F		77	63
Residue from distillation to 680°F, Volume % by diffence		72	81
Penetration on residue at 77°F, 100 gm., 5 sec. (ASTM Method D 5)		239	313

Preparation and Mixing of Soils

All soil brought to the laboratory for testing was stored in barrels. Soil to be used the following day in testing was passed through a No. 4 sieve, placed in a large pan, and allowed to air-dry. Clay lumps were broken down to pass the No. 4 sieve and thoroughly mixed in with the soil while roots were discarded.

The constituents of the mixture were blended with a Hobart C-100 mixer equipped with a flat blade. The mixture was blended at a speed of 144 revolutions per minute. After mixing, the blended material was aerated in 12 x 24 x 3 in. metal trays. Air was circulated across these trays using an Emerson Electric fan with a 16 in. blade. These two items of equipment are shown in use in Fig. 8.

Before and after mixing, as well as during the aeration process, approximate moisture contents were taken with a Speedy Moisture Tester manufactured by the Alpha-Lux Company. More accurate moisture contents were determined by oven-drying samples at 110°C for 24 hours.

Moisture-Density Tests

The equipment used, and the general procedure followed in the moisture-density tests for the rapid curing asphalts conform to the AASHO T99 Compaction Test and for the medium curing asphalts, conform to the AASHO T180-57.

For each point on the moisture-density curve five lbs. of dry soil were combined with a predetermined amount of water. Next, the correct increment of cutback asphalt was combined with the soil and water and mixed thoroughly. The prescribed compaction effort was applied, wet density determined and a representative sample of the mixture was placed in an oven at 110°C for 24 hours for liquid content determination. The liquid content of each test point was then compared to the water content of the same test points so as to determine the correction factor relating liquid and water contents for this particular soil and increment of cutback asphalt. Optimum moisture content

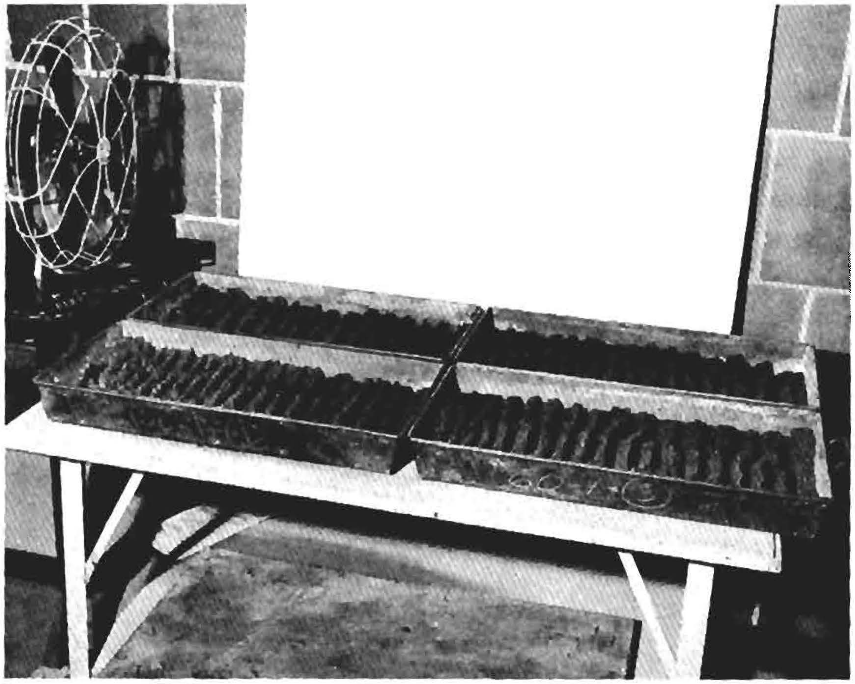

Figure 8. Aeration of Soil Bituminous Mixtures Prior to Compaction.

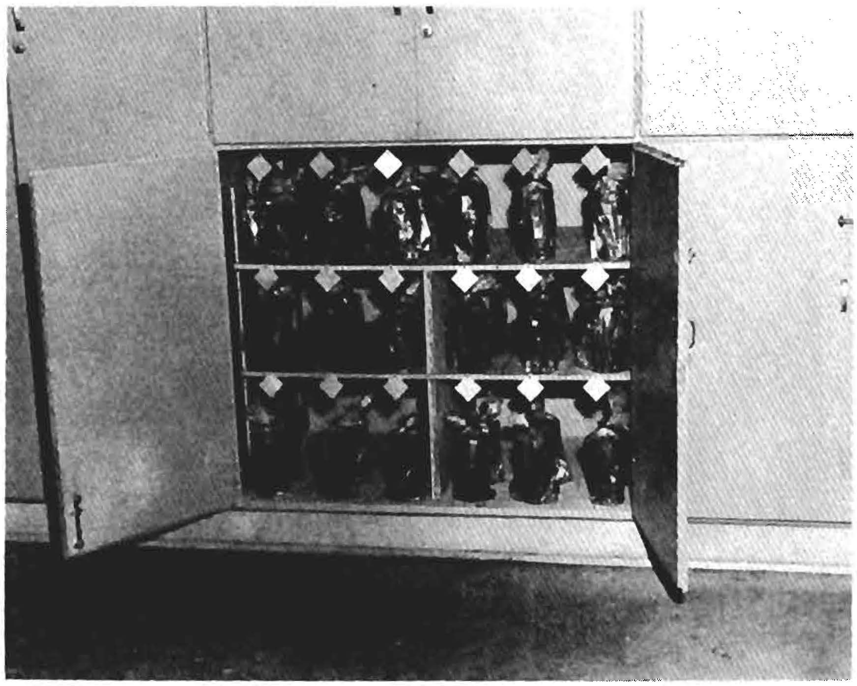

Figure 9. Storage Cabinet for Triaxial Shear Strength Specimens.

of the soil was found by applying the correction factor to the liquid content corresponding to maximum dry density.

Molding Test Specimens

The test specimens were 2.8 inches in diameter and 5.6 inches high. The molding procedure and apparatus used in these tests was similar to the procedure and apparatus used in Chapter II.

Curing

A stabilized base or subgrade in a modern highway construction project is only one component of many constituting the final cross section. It can be reasoned from this that time is of essence in curing stabilized bases and subgrades so as to allow the application of the remaining components of the highway structure. Applying one of these components such as a concrete or bituminous wearing course virtually seals the stabilized material from exposure to the elements, with the exception of normal fluctuations of ground water.

In keeping with a principle discussed in the beginning of this chapter, the curing effort applied to test specimens in this research was selected so as to closely approximate the construction techniques of field curing. After compaction, triaxial shear strength samples were sealed in polyethylene freezer bags so as to minimize moisture loss. These sealed samples were then stored in a cabinet for periods up to 7 days. Samples with 3, 5, and 7 per cent RC-3 were also stored 28 days.

No attempt was made to regulate temperatures or humidity within the storage cabinet. The curing cabinet complete with triaxial shear strength samples is shown in Fig. 9.

Triaxial Shear Strength Testing

Confined and unconfined compressive strength evaluations were made on each soil and each test increment of RC-3 for both maximum and dry-back stages. Lateral pressure equal to 20 psi was developed by introducing compressed air into a sealed plexiglass cylinder. Thin rubber membranes were placed around confined compressive strength samples.

Strain measurements were recorded in increments of 0.025 inches using a Ames dial attached to the triaxial cell.

The rate of loading corresponded to 0.75 inches per minute of vertical head travel. After completion of the triaxial shear test a moisture content sample was removed from each test cylinder.

Fig. 4 shows a triaxial shear test in progress.

Mixing and Compaction of Triaxial Shear Strength Cylinders

RC-3 was added to each soil in increments of 2, 3, 4, 5, 6, and 7 per cent total cutback asphalt by weight of dry soil. Consideration was given to heating the RC-3 prior to mixing with the soil. However, when hot cutback asphalt was introduced to a soil at room temperature, the mixture would coagulate before thorough mixing was accomplished. Consequently, soil and RC-3 were combined at room temperature to facilitate the distribution of cutback asphalt within the soil material.

Several sequences of mixing were investigated. Optimum results were obtained by simultaneously introducing RC-3 and water to the soil. At the time of introduction, the air-dry soil was being agitated by the mixer at a speed of 144 revolutions per minute. Mixing was then continued at this speed with frequent stops to remove material from the beater and to prevent caking around the sides of the mixing bowl. Total elapsed time for the mixing phase was 10 minutes.

Medium Curing

Cutback asphalt was added to Soil X and XI as a percentage of dry soil. For example, a mix containing "2 per cent MC-2" would be composed of 10 pounds of dry soil and 0.2 pound of MC-2. The percentages of MC-2 and MC-4 used were 1, 2, 3, 4, 5, and 6.

Test Results

Moisture-Density Relationships for Soil-Asphalt Mixes

It has been found that the drying, before compaction, of mixes of soil and cutback asphalt may have a significant effect on the density of the compacted mix.[9,10] This effect is related to the evaporation of hydrocarbon volatiles in the cutback asphalt and the evaporation of water. The influence of the individual losses is not known.

Since it is generally believed that the asphalt volatile loss causes an increase in the strength of a compacted soil-asphalt base course, soil asphalt mixes are usually allowed to "dry back" before being compacted. The length of this drying period varies; for example, some states specify a minimum drying period while other states leave this to the judgement of the engineer-in-charge. It should be mentioned here that the effects of air temperature and humidity on the degree of volatile loss is usually neglected in determining the length of the drying period to be used in the field. Also, it is interesting to note that researchers in the past have ignored these two important variables.

Because of the appreciable effect of drying the mix, moisture-density relationships were determined for mixes dried back for 3 days and 6 days before compaction as well as for mixes compacted immediately following mixing. The latter drying period, although, perhaps too long for practical purposes, was established so that the effect of such extensive drying could be determined. Other investigators[11] have used shorter periods of time for oven-drying mixtures before compaction. It is believed that good correlation between oven-drying and air-drying is difficult to obtain. For this reason, all drying of mixes was done at air temperatures in the approximate range of 65-85°F.

The results of the moisture-density determinations using RC-3 are presented in tabular form in Table VII. The information contained in this table includes maximum dry density (M.D.D.), optimum liquid content (O.L.C.) and optimum moisture content (O.M.C.) for each soil combined with 2, 3, 4, 5, 6, and 7 per cent RC-3. Any blank columns in this table indicates an insufficient quantity of material to complete the moisture-density series.

The overall effect of the addition of RC-3 on the moisture-density characteristics of a particular soil can be evidenced in the following summary table.

Soil No.	AASHO Designation	Maximum Dry Density	Optimum Moisture Content
I	(A-4-(0)	decreased	decreased
II	(A-1-b(0))	increased then decreased	increased
III	(A-3-(0))	increased then decreased	increased
IV	(A-4-(4))	decreased	decreased
VI	(A-4(2))	decreased	increased
VII	(A-4(1))	decreased	decreased
VIII	(A-7-5(15))	increased then decreased	constant
IX	(A-5(8))	decreased	constant

An inspection of the preceding tabulation indicates that no broad statement can be made concerning the effect of RC-3 on the moisture-density characteristics of a soil material. However, by correlating the variation in maximum density with the gradation characteristics of each soil a significant relationship is obtained. Each soil that increased in density (II, III, and VIII) has the same general gradation characteristics (uniformity). A uniform gradation indicates that the majority of the individual particles have much the same size. Therefore, certain particle sizes required to fill voids created by larger particles are lacking. This leads to low densities for uniformly graded materials when compared with well-graded soils. The extent to

Table VII. Maximum Dry Density, Optimum Liquid Content and Optimum Moisture Content for Soils I, II, III, IV, VI, VII, VIII, and IX Combined with 2, 3, 4, 5, 6 and 7% RC-3

RC-3	0%		2%			3%		4%			5%		6%			7%	
Soil	M.D.D. $\#/ft^3$	O.M.C. %	M.D.D. $\#/ft^3$	O.L.C. %	O.M.C. %	M.D.D. $\#/ft^3$	O.M.C. %	M.D.D. $\#/ft^3$	O.L.C. %	O.M.C. %	M.D.D. $\#/ft^3$	O.M.C. %	M.D.D. $\#/ft^3$	O.L.C. %	O.M.C. %	M.D.D. $\#/ft^3$	O.M.C. %
I	121.2	12.0	120.0	11.3	10.8	123.0	8.7	118.4	11.0	10.3	123.2	8.4	116.6	10.5	9.8	123.0	6.4
II	119.1	10.3	122.1	10.4	9.9	121.9	9.5	121.5	9.5	9.0	121.5	8.0	121.2	8.5	8.0	119.1	8.5
III	101.0	9.5	104.2	12.0	11.7	107.2	12.5	106.8	11.0	10.7	107.4	11.0	--	--	--	109.0	10.0
IV	114.2	14.6	113.1	14.6	14.2	114.5	14.0	112.7	13.5	13.0	114.1	12.3	111.9	13.5	13.0	113.2	12.6
VI	110.2	14.7	106.8	15.6	15.2	113.3	13.5	108.4	13.8	13.2	112.3	13.7	107.5	12.7	12.0	111.5	13.7
VII	117.4	13.0	113.4	13.5	13.0			112.1	12.5	11.9			110.4	13.5	12.8		
VIII	88.7	30.9	91.4	26.1	25.5			92.3	25.8	25.5			90.4	26.5	25.8		
IX	100.4	22.4	96.8	23.5	23.0			94.0	24.0	23.5			94.5	23.0	22.6		

which this uniformity affects density would depend on the diameter of the uniform particles, with the effect diminishing as particle size decreases.

Hence, any admixture that could serve to fill these voids would in effect contribute to the density of the combination of materials.

Conversely, the addition of cutback asphalt to a well graded material such as Soils I, IV, VI, VII, and IX could decrease maximum density by preventing intimate grain contact of soil particles.

In summary, well-graded soils showed a decrease in density while the density of uniformly graded soils was increased by the addition of RC-3. In no instance was the deviation between these two strength values of enough significance to warrant rejection.

Confined and unconfined compressive strength values for maximum stage samples are presented in tabular form in Table VIII. Graphic illustration of the variation in strength with increasing RC-3 content for maximum stage samples is presented in Figures 10 through 16.

If RC-3 is combined with a soil and compacted at maximum density and optimum moisture content, no strength gains of significance will result. In the majority of soils tested, compressive strength remained constant, or in some cases decreased slightly. The exception to this situation was Soil VIII which showed an increase in strength of 50 per cent when combined with RC-3 and compacted at maximum density and optimum moisture content.

Two types of soils (Soils X and XI) were utilized in the medium-curing cutback asphalt stabilization studies. Soil X was used in a pilot study to determine the effect of MC-2 temperatures on moisture-density. The purpose of this study was to determine the waterproofing properties of MC-2 and MC-4 cutback asphalt and the effect of this asphalt on strength. Some of the variables considered were asphalt temperatures at the time of introduction to

Table VIII. Confined and Unconfined Compressive Strength Values (psi) for Maximum Stage Samples

RC-3(%)	0		2		3		4		5		6		7	
Soil	0*	20	0	20	0	20	0	20	0	20	0	20	0	20
I	32.0	62.0	38.0	64.5	14	53	39.4	54.1	19	67	30.0	46.0	26	71
II	15.0	85.2	10.0	72.0	15	63	13.0	61.0	30	75	14.0	64.0	24	53
III	--	--	--	--	2	52	--	--	2	60	1.1	43.0	3	69
IV	33.0	58.0	26.3	47.0	34	51	32.4	61.4	37	54	31.0	55.0	42	48
VI	37.0	89.0	19.4	32.9	32	55	34.0	86.3	31	57	35.9	91.9	40	65
VII	42.0	68.0	33.8	60.2			52.4	83.6			44.0	65.7		
VIII	47.0	56.2	49.1	84.5			48.0	83.2			47.0	73.2		
IX	32.0	56.0	35.1	61.8			34.9	63.7			36.0	46.2		

*Note: 0 and 20 indicate confining pressure (σ_3) in psi.

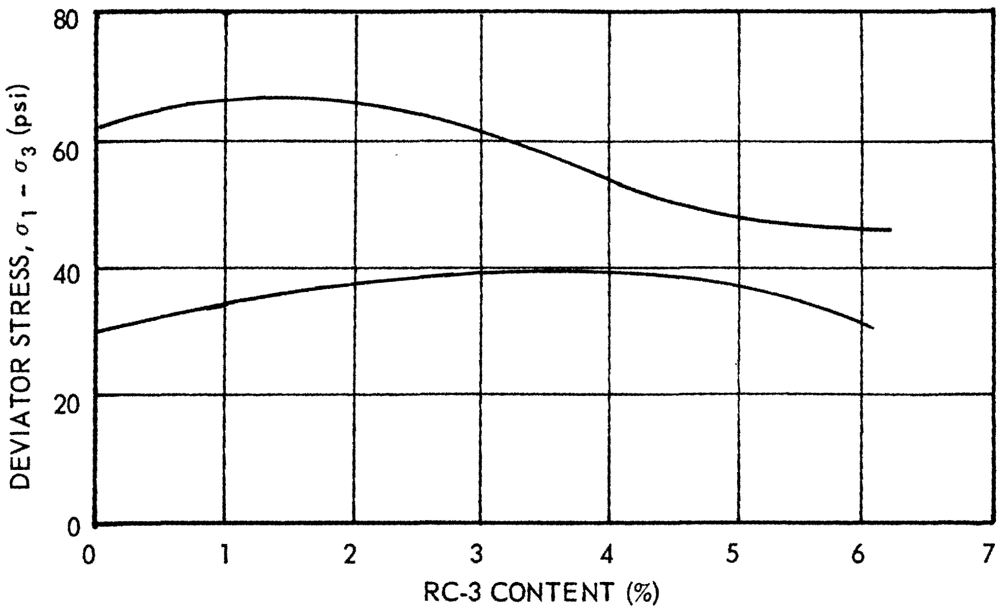

Figure 10. Variation in Deviator Stress with Increasing RC-3 Content, Soil I. Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

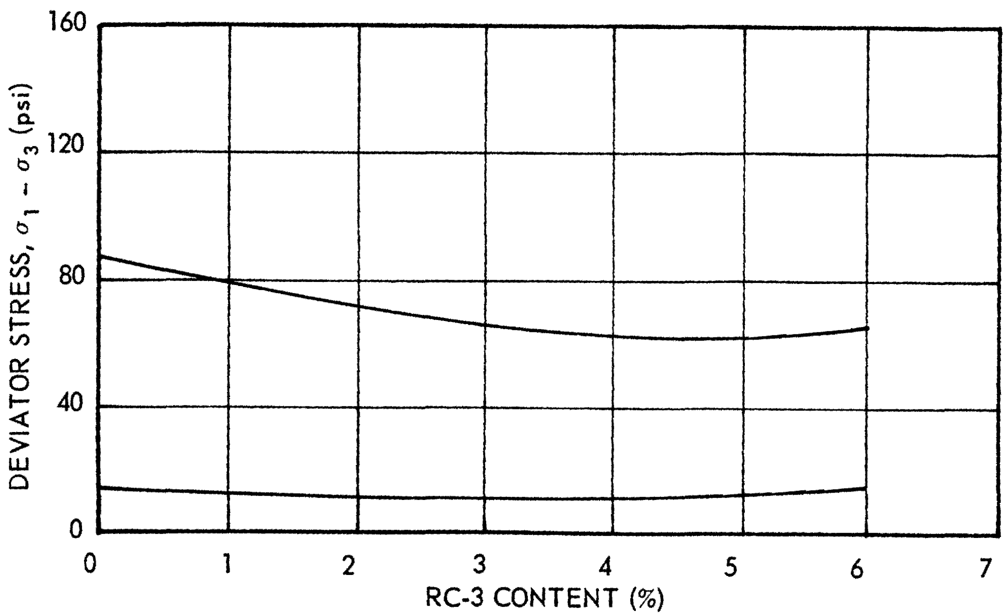

Figure 11. Variation in Deviator Stress with Increasing RC-3 Content, Soil II. Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

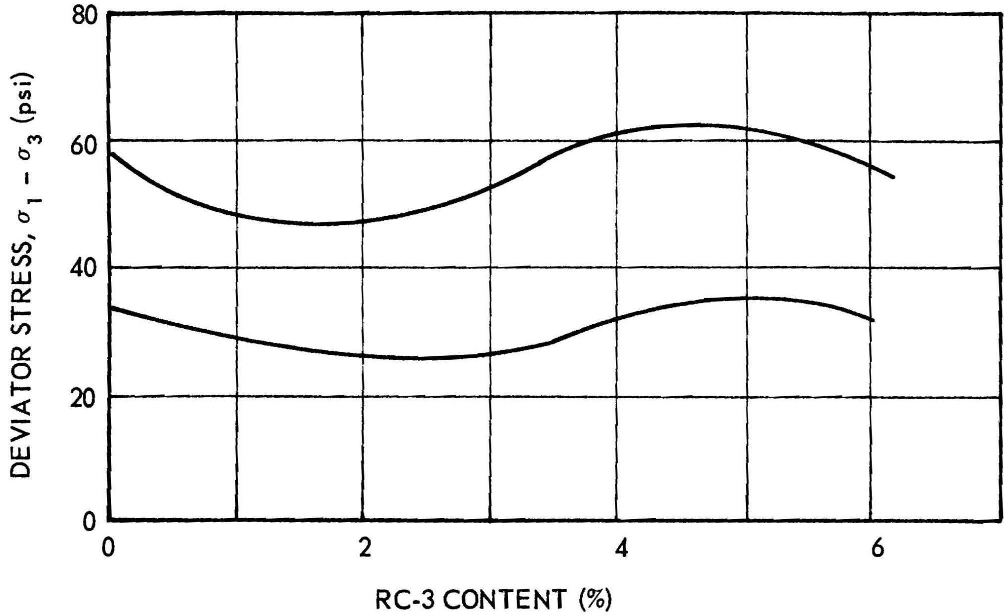

Figure 12. Variation in Deviator Stress with Increasing RC-3 Content, Soil IV. Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

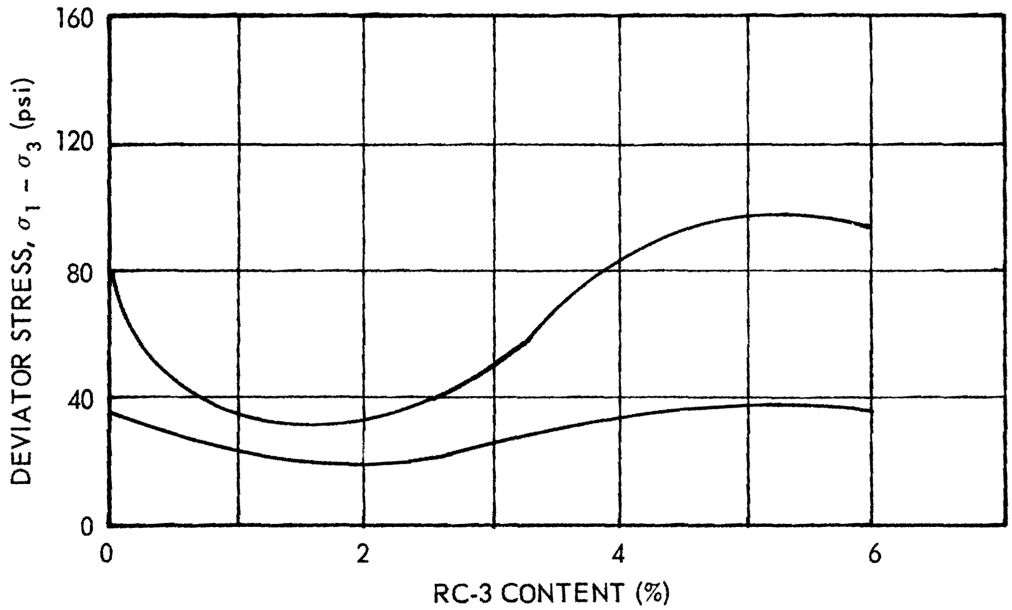

Figure 13. Variation in Deviator Stress with Increasing RC-3 Content, Soil VI. Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

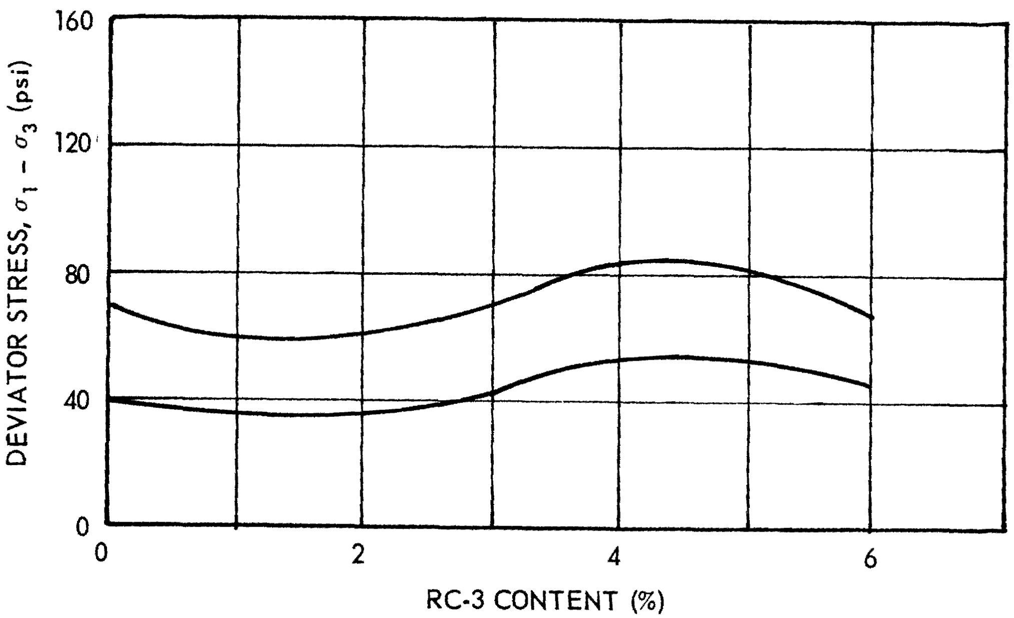

Figure 14. Variation in Deviator Stress with Increasing RC-3 Content, Soil VII. Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

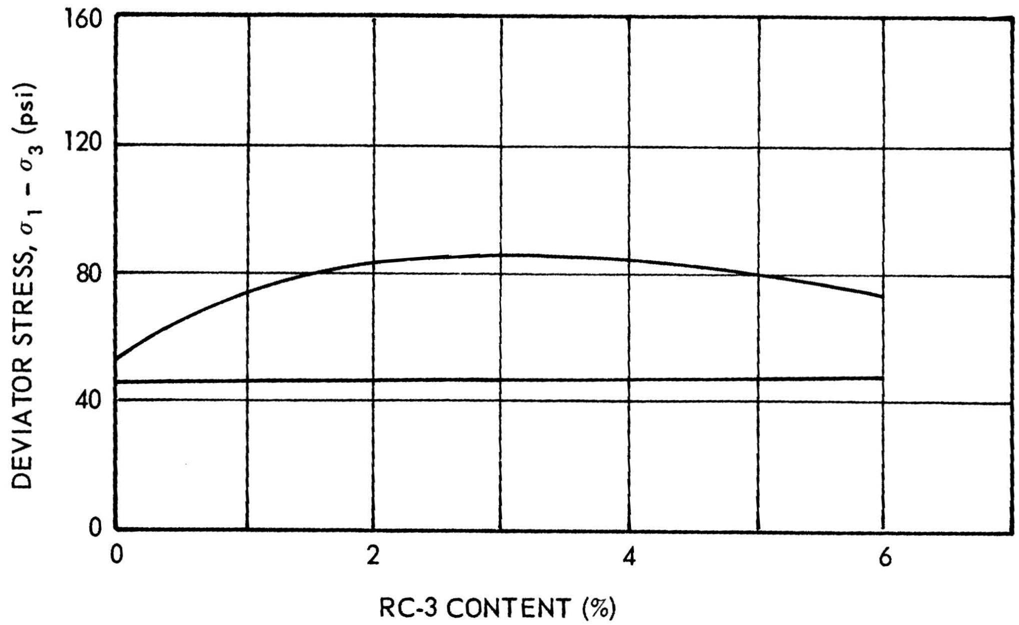

Figure 15. Variation in Deviator Stress with Increasing RC-3 Content, Soil VIII. Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

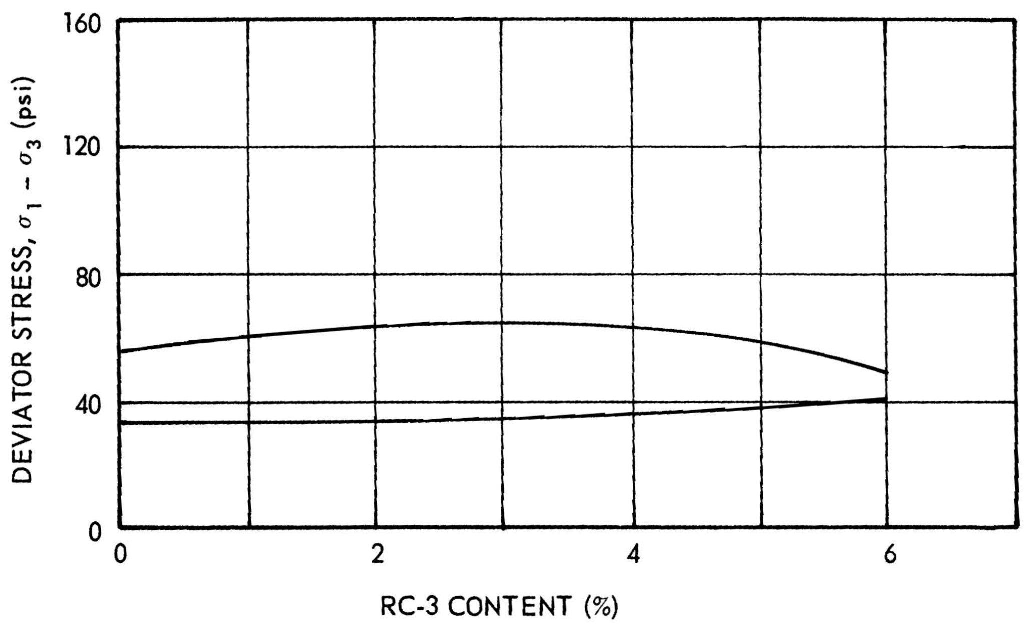

Figure 16. Variation in Deviator Stress with Increasing RC-3 Content, Soil IX, Top Curve Q_3 = 20 psi, Bottom Curve Q_3 = 0.

soil mixing time, and drying time before compaction.

The moisture-density relationships were determined for Soil X and XI combined with their respective percentages of MC-2 and MC-4 cutback asphalt ranging from 0 to 6 per cent. This was done for drying periods before compaction of 0,3 and 6 days. These results are given in Table IX.

MC-4 was added to Soil XI at seven different percentages (0 - 6) and each percentage at three different temperatures (100°, 125° and 150°F.). The moisture-density date for these combinations are in Table X.

Compressive Strength

By utilizing values obtained from the moisture-density tests, triaxial shear strength specimens 2.8 inches in diameter and 5.6 inches in height were statically compacted to both maximum density and optimum moisture content and to densities less than maximum with corresponding moisture contents. The purpose in compacting samples at less than maximum density was to investigate the relationship existing between density and shear strength in bituminous stabilized soil materials.

After compaction and prior to testing, triaxial shear strength specimens were sealed in polyethylene bags and stored at room temperature for one week. Samples with 3, 5, and 7 per cent RC-3 were also stored 28 days. Strength evaluations were made from unconfined compressive strength tests and triaxial shear strength tests with a lateral pressure of 20 psi. These results are given in Table VIII.

Testing was employed for various combinations of Soil XI, water and MC-2 cutback asphalt; whole percentages of asphalt, from 0 to 6 were used. Three variables were considered for each asphalt content used, drying time between mixing and compaction, curing time after compaction, and soaking time between curing and strength testing. Soaking was accomplished by the apparatus shown

Table IX. Maximum Dry Densities for Soil XI Combined With MC-2 Cutback Asphalt, Varying Drying Period

Per Cent MC-2	Drying Period (Days) 0		3		6	
	M.D.D.	O.M.C.	M.D.D.	O.M.C.	M.D.D.	O.M.C.
0	122.4	9.4	-	-	-	-
1	123.8	7.8	126.3	14.7	124.2	10.6
2	126.1	6.5	126.5	13.6	124.8	10.8
3	124.8	6.0	126.6	12.5	124.9	10.1
4	125.5	4.7	125.5	11.8	124.4	7.7
5	125.2	4.1	125.4	10.9	124.6	6.8
6	125.6	2.8	125.2	7.0	123.9	6.0

Table X. Moisture-Density Data for Soil XI Combined With MC-4; Varying Mixing Temperature of MC-4; No Drying

	Mixing Temperature of MC-4					
	100°F		125°F		150°F	
Per Cent MC-4	M.W.D.	O.M.C.	M.W.D.	O.M.C.	M.W.D.	O.M.C.
0	133.9	9.4	133.9	9.4	133.9	9.4
1	132.1	8.6	135.2	7.7	134.6	8.1
2	135.7	7.1	135.5	7.1	135.2	6.9
3	136.4	6.3	135.8	6.2	136.6	6.0
4	137.5	5.5	137.5	5.6	136.9	4.7
5	137.1	4.9	137.0	4.7	136.8	5.0
6	136.8	3.9	136.8	4.0	136.3	3.2

in Fig. 17. Strength testing consisted of performing triaxial shear tests, using a confining pressure of 20 psi. These results are given in Table XI.

Conclusion

The addition of cutback asphalt to the soils used in this research did not materially increase the strength parameters of these soils. However, it can be concluded from an analysis of the test results that the maximum strength properties of each soil do not necessarily occur at the density and moisture content corresponding to the peak of the moisture density curve.

The asphalt content that had optimum influence on the strength parameters of the soils tested varied but was generally found in the range of 2 to 4 per cent.

In combining the materials it was evident that most intimate mixing of soil and RC-3 occurred at a moisture content at or near optimum moisture content.

It was also concluded from the series of tests that the drying mixtures of Soil XI and MC-2, up to 6 days before compaction, resulted in higher dry densities than obtained by compacting identical mixtures immediately following mixing; the temperatures of the MC-4 at the time it was introduced to Soil XI had little influence on the densities of the compacted mixture; for each asphalt content there is a unique combination of drying-back before compaction and curing after compaction which yields the most stable sample under a certain condition of soaking.

Table XI. Summary of Compressive Strength Test Results

No. of Days of			MC-2 Content 0%		MC-2 Content 1%		MC-2 Content 2%		MC-2 Content 3%		MC-2 Content 4%		MC-2 Content 5%		MC-2 Content 6%	
Drying Back	Curing	Soaking														
0	0	0	122.4	60	123.8	79	126.1	85	124.8	83	125.5	86	125.2	86	125.6	87
3	0	0			124.2	86	124.8	88	124.9	86	124.6	86	124.4	87	123.9	87
6	0	0			126.3	88	126.5	97	126.6	86	125.5	72	125.4	78	125.2	82
0	0	3	122.4	27	123.8	23	126.1	32	124.8	50	125.5	50	125.2	63	125.6	52
3	0	3			124.2	26	124.8	34	124.9	52	124.6	66	124.4	72	123.9	74
6	0	3			126.3	28	126.5	33	126.6	67	125.5	78	125.4	73	125.2	76
0	3	0	122.4	72	123.8	79	126.1	86	124.8	70	125.5	81	125.2	82	125.6	78
3	3	0			124.2	84	124.8	84	124.9	89	124.6	83	124.4	85	123.9	83
6	3	0			126.3	85	126.5	93	126.6	83	125.5	79	125.4	74	125.2	68
0	3	3	122.4	64	123.8	38	126.1	52	124.8	19	125.5	33	125.2	66	125.6	69
3	3	3			124.2	10	124.8	8	124.9	81	124.6	89	124.4	63	123.9	46
6	3	3			126.3	70	126.5	94	126.6	87	125.5	80	125.4	71	125.2	84

= Maximum Dry Density (lb./cu. ft.)

= Normal Stress (psi) @ σ_3 = 20 psi

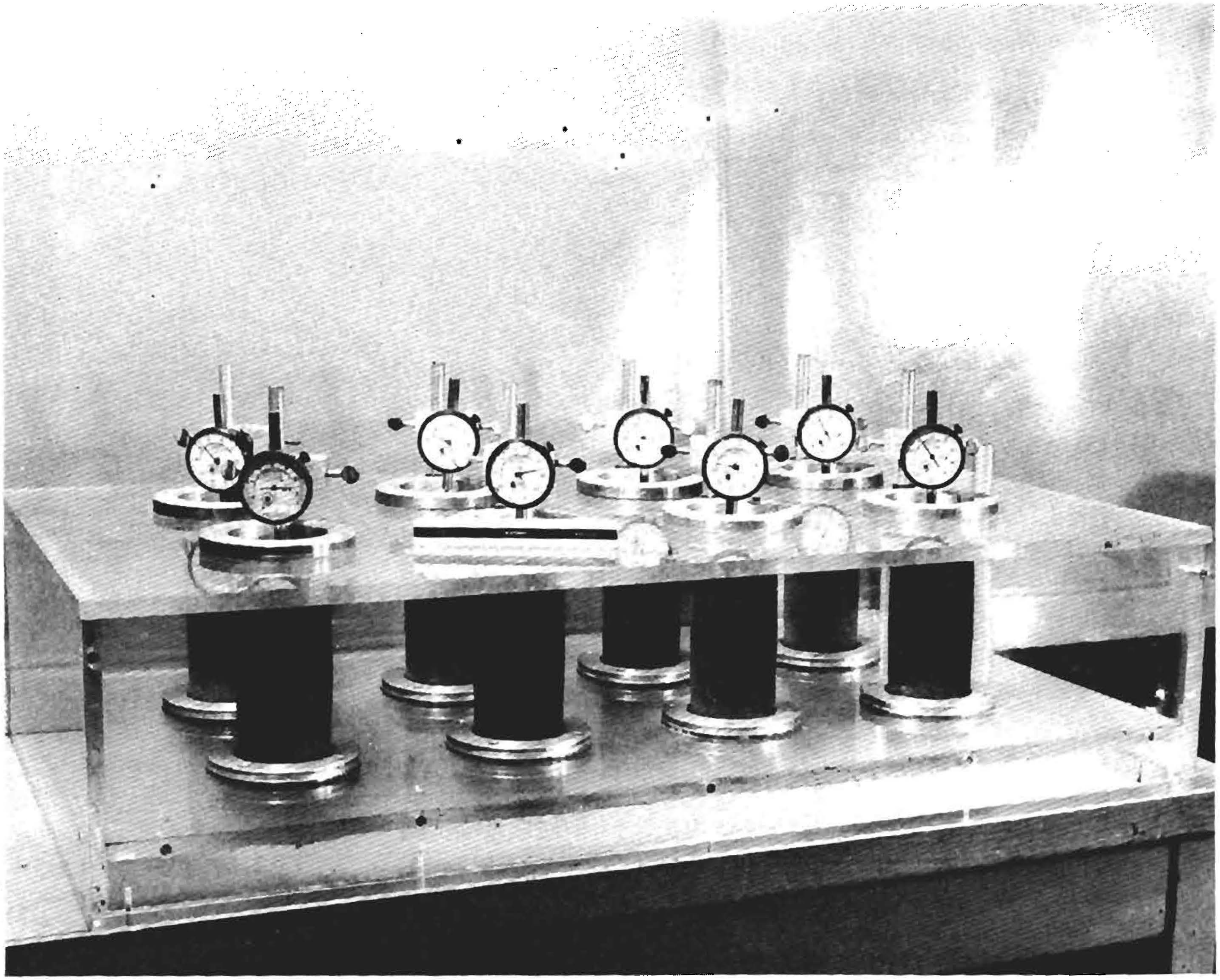

Figure 17. Samples being Soaked before Triaxial Shear Testing.

CHAPTER IV

SOIL, STONE SCREENINGS AND CEMENT

Materials Used and Test Methods

Soils Used

Five different soils were used with stone screenings and cement. These soils are designated as IV, V-A, VII, VIII, and IX. All the soils, with the exception of Soil V-A, were used in previous research on this project.

The determination of the liquid limit, plastic limit, specific gravity, and grain size distribution for each of these soils was done according to the standard methods of test of the American Association of State Highway Officials.[10] The values obtained are shown in Table I, along with other pertinent data on the soils.

Proportioning of Soil, Stone Screenings, and Cement

Stone screenings were added to each of Soils IV, V-A, VII, VIII, and IX as a percentage of total weight of dry soil plus screenings. For example, a 10-pound batch of soil and screenings said to contain 25 per cent screenings was composed of 7.5 pounds of dry soil and 2.5 pounds of screenings.

For all the above soils except Soil IX, the addition of portland cement was based on a percentage of total weight of dry soil plus stone screenings. In other words, a typical batch containing "25 per cent screenings plus 2 per cent cement" would consist of 7.5 pounds of dry soil, 2.5 pounds of screenings, and 0.2 pounds of cement. Portland cement was added to Soil IX, the first soil tested, as a percentage of dry soil only. In this case, "25 per cent screenings plus 2 per cent cement" meant 7.5 pounds of dry soil, 2.5 pounds of screenings and 0.15 pounds of cement. In order to compare the effects of these admixtures on the different soils, it was necessary to adjust the data obtained in the tests utilizing Soil IX.

The screenings contents used were 0, 25, 50, and 75 per cent. The cement percentages were 0, 2, 4, 8, and 12. This resulted in 20 different combinations of soil, stone screenings and cement for each soil.

Moisture-Density Relationship for Soil, Stone Screenings, and Cement

Moisture-density relationships were determined for each of the 20 combinations described above and are given in Table XII.

The procedure used for preparing and compacting a moisture-density sample was as follows:

1. The hygroscopic moisture of the soil was determined with a "Speedy" Moisture Tester manufactured by the Alpha Lux Co., Inc. The amount of water to be added to attain the desired water content was then found.
2. The desired amounts of soil, screenings, cement, and water were weighed to the nearest 0.1 pound.
3. The components of the mix less the water were placed in a 10-quart capacity mixing bowl and mixed manually until a fairly uniform mix was obtained.
4. The ingredients were then mixed mechanically by a Hobart C-100 mixer with a flat blade. After the mixer had been running for a few seconds, the water was added. All ingredients were mixed for a total of ten minutes, the mixing being interrupted once or twice to scrape the blade and the inside of the mixing bowl clean.
5. The mix was compacted according to the method outlined in AASHO Designation: T 99-57.

Table XII. Moisture-Density Data for Soils IV, V-A, VII, VIII, and IX

Percent Screenings	Percent Cement	Soil IV M.D.D.*	Soil IV O.M.C**	Soil V-A M.D.D.	Soil V-A O.M.C.	Soil VII M.D.D.	Soil VII O.M.C.	Soil VIII M.D.D.	Soil VIII O.M.C.	Soil IX M.D.D.	Soil IX O.M.C.
0	0	114.2	14.6	95.7	20.3	116.1	14.0	88.7	30.9	100.4	22.4
0	2	112.4	15.5	94.2	25.0	114.8	13.9	90.3	30.7	101.9	22.0
0	4	111.6	16.6	94.9	23.9	115.2	14.2	90.4	30.3	101.0	21.7
0	8	112.3	15.8	95.8	24.0	116.1	13.8	90.0	30.9	101.5	21.5
0	12	114.2	15.1	96.2	23.4	117.3	13.4	90.6	30.5	103.3	20.5
25	0	118.8	13.0	102.7	21.2	121.0	11.5	96.6	24.6	108.0	18.0
25	2	115.4	14.4	102.8	21.0	120.1	12.0	98.0	24.0	109.1	17.6
25	4	117.1	13.9	105.6	19.8	120.7	12.1	98.5	23.5	109.2	17.3
25	8	118.2	13.0	106.0	19.8	121.0	11.9	100.0	23.5	109.3	17.7
25	12	118.0	13.0	105.7	19.4	121.8	11.6	100.1	23.4	109.3	18.6

*M.D.D. = Maximum Dry Density

**O.M.C. = Optimum Moisture Content

Table XII. Moisture-Density Data for Soils IV, V-A, VII, VIII, and IX (Cont.)

Percent Screenings	Percent Cement	Soil IV M.D.D.*	Soil IV O.M.C.**	Soil V-A M.D.D.	Soil V-A O.M.C.	Soil VII M.D.D.	Soil VII O.M.C.	Soil VIII M.D.D.	Soil VIII O.M.C.	Soil IX M.D.D.	Soil IX O.M.C.
50	0	126.3	10.9	114.3	14.6	125.2	9.5	107.7	18.7	115.0	14.0
50	2	125.3	10.9	114.7	13.1	123.5	10.8	108.0	18.5	115.8	14.5
50	4	124.0	11.9	115.0	14.7	123.7	10.7	108.6	18.6	114.8	14.5
50	8	125.1	10.6	115.3	14.6	124.6	10.5	109.3	17.7	115.0	14.3
50	12	126.5	10.8	114.2	13.2	125.0	10.5	109.7	17.5	117.0	14.0
75	0	132.5	8.2	126.7	9.6	126.0	10.0	121.1	11.2	122.8	10.2
75	2	132.6	8.8	126.3	9.3	126.3	9.4	120.2	11.5	124.0	10.5
75	4	133.5	8.8	126.8	10.5	127.2	9.2	122.8	11.1	125.6	10.5
75	8	133.8	8.6	126.8	9.5	129.5	8.9	122.5	11.3	122.8	10.5
75	12	132.4	8.8	128.0	9.7	130.0	8.8	122.7	11.8	125.0	10.1

*M.D.D. = Maximum Dry Density

**O.M.C. = Optimum Moisture Content

At least six different water contents were used for each combination of soil, screenings, and cement. After a mix had been compacted, it was not used again.

Determination of Compressive Strength of Compacted Specimens of Soil, Stone Screenings, and Cement

The compressive strength was determined for each combination of soil, stone screenings, and cement, utilizing the optimum water content and the corresponding maximum dry density. Tabular results are given in Tables XIII through XVII.

Eight samples, 2.8 inches in diameter and 5.6 inches high, were molded for each combination. Four of these samples were cured for 7 days prior to testing while the remaining four were cured for 28 days.

The procedure used for preparing the strength-test specimens was as follows:

1. A calculation was made of the amount of each ingredient necessary to yield a batch sufficient for four molded specimens at the maximum dry density and optimum moisture content. The calculated batch weights were then increased enough to provide two moisture samples of about 100 grams each.
2. The adjusted amount of each ingredient was weighed to the nearest 0.1 pound and the constituents blended and mixed in the same manner as used for the moisture-density mixes.

Molding Test Specimens

Molding of the specimens was begun as soon after completion of the mixing process as possible. The equipment used for molding the specimens is shown in Figures 1 and 2. The procedure followed in the molding and extruding of the compressive-strength samples was the same as used in Chapter II.

Table XIII. Summary of Compressive Strength Tests, Soil IV Combined With Stone Screenings and Portland Cement

		Normal Stress σ_1 (psi)			
Length of Cure (days)		7	28	7	28
Lateral Pressure, σ_3 (psi)		20	20	0	0
% Screenings	% Cement				
0	0	58.0	73.0	33.0	38.0
0	2	137	169	104	109
0	4	270	347	241	284
0	8	333	493	327	387
0	12	457	588	460	539
25	0	29.9	48.4	19.1	23.9
25	2	189	246	148	199
25	4	429	557	362	501
25	8	611	887	528	755
25	12	724	989	590	823
50	0	50.0	60.3	16.8	18.8
50	2	287	333	206	258
50	4	418	557	376	464
50	8	656	742	515	589
50	12	882	1048	757	830
75	0	73.5	68.0	9.5	11.0
75	2	289	375	157	238
75	4	438	498	292	360
75	8	684	818	525	635
75	12	906	1058	756	850

Table XIV. Summary of Compressive Strength Tests, Soil V-A Combined With Stone Screenings and Portland Cement

		Normal Stress σ_1 (psi)			
Length of Cure (days)		7	28	7	28
Lateral Pressure σ_3 (psi)		20	20	0	0
% Screenings	% Cement				
0	0	59.5	74.2	23.6	35.0
0	2	42.0	69.0	22.5	33.0
0	4	130	166	80.1	114
0	8	385	523	314	531
0	12	554	843	496	762
25	0	33.0	39.0	16.0	17.0
25	2	50.0	73.0	29.0	42.0
25	4	237	271	149	238
25	8	544	789	438	709
25	12	723	1067	623	968
50	0	55.0	48.0	20.0	17.0
50	2	130	169	70.0	108
50	4	346	477	291	418
50	8	666	866	621	789
50	12	598	888	501	762
75	0	61.0	66.0	12.9	15.0
75	2	222	321	141	230
75	4	382	718	209	653
75	8	496	811	360	675
75	12	722	1068	619	912

Table XV. Summary of Compressive Strength Tests, Soil VII Combined With Stone Screenings and Portland Cement

		Normal Stress σ_1 (psi)			
Length of Cure (days)		7	28	7	28
Lateral Pressure σ_3 (psi)		20	20	0	0
% Screenings	% Cement				
0	0	63.7	74.2	33.6	53.8
0	2	190	209	169	200
0	4	421	555	393	550
0	8	731	909	737	866
0	12	944	1075	932	1023
25	0	70.0	67.7	20.7	23.7
25	2	242	312	186	264
25	4	430	653	362	599
25	8	652	945	527	872
25	12	958	1316	819	1124
50	0	81.1	80.0	14.7	15.5
50	2	255	329	153	266
50	4	386	547	288	476
50	8	697	980	573	843
50	12	1009	1179	838	1084
75	0	73.3	74.7	4.6	3.7
75	2	236	290	103	166
75	4	378	377	236	252
75	8	806	824	630	648
75	12	1305	1682	1158	1558

Table XVI. Summary of Compressive Strength Tests, Soil VIII Combined With Stone Screenings and Portland Cement

		Normal Stress σ_1 (psi)			
Length of Cure (days)		7	28	7	28
Lateral Pressure, σ_3 (psi)		20	20	0	0
% Screenings	% Cement				
0	0	56.2	58.0	47.0	38.0
0	2	92.0	83.0	71.0	56.0
0	4	147.5	137.0	95.0	100.0
0	8	280.0	291.0	242.0	242.0
0	12	341.0	444.0	281.0	395.5
25	0	60.7	45.0	40.0	37.6
25	2	95.2	88.0	73.0	62.0
25	4	166.0	202.0	148.0	169.0
25	8	349.2	394.0	278.6	346.0
25	12	434.0	568.0	430.0	539.7
50	0	39.1	45.2	21.8	31.2
50	2	190.0	266.0	160.0	225.0
50	4	417.0	581.0	375.0	518.0
50	8	527.0	847.0	466.0	776.0
50	12	649.0	945.0	555.0	923.0
75	0	77.6	80.0	29.6	37.0
75	2	324.0	458.0	250.0	382.0
75	4	506.0	744.0	432.0	664.0
75	8	695.0	1032.0	632.0	882.0
75	12	972.0	1392.0	848.0	1242.0

Table XVII. Summary of Compressive Strength Tests, Soil IX Combined With Stone Screenings and Portland Cement

		Normal Stress σ_1 (psi)			
Length of Cure (days)		7	28	7	28
Lateral Pressure, σ_3 (psi)		20	20	0	0
% Screenings	% Cement				
0	0	56.0	106.5	32.0	71.7
0	2	83.5	92.5	57.8	62.0
0	4	197.3	216.5	138.0	194.0
0	8	336.8	382.0	288.8	338.5
0	12	423.3	571.0	356.5	502.0
25	0	57.8	130.1	41.8	81.0
25	1.5	123.1	280.0	98.5	98.2
25	3	295.3	416.6	256.1	354.1
25	6	513.0	718.3	466.1	651.8
25	9	601.6	1065.2	630.5	1060.5
50	0	78.6	40.9	42.4	38.3
50	1	95.5	95.5	80.9	93.2
50	2	271.5	391.3	252.0	373.7
50	4	379.0	691.0	401.0	637.0
50	6	456.0	755.0	462.0	792.0
75	0	30.9	80.4	27.8	38.8
75	0.5	72.6	95.5	35.2	86.5
75	1.0	221.0	280.0	140.0	213.0
75	2.0	310.0	442.0	235.0	369.0
75	3.0	357.0	588.0	288.0	525.0

Triaxial Compressive Test

At the end of the prescribed curing period, the samples were tested for compressive strength. Two of the samples were tested without confining pressure and the other two were subjected to a lateral pressure of 20 pounds per square inch during the loading period. Figure 4 shows a triaxial shear test in progress.

After the sample was removed from the polyethylene bag, it was placed in the triaxial cell. For confined samples, a thin rubber membrane was placed over the sample so that the air pressure would not be exerted on the soil itself. The top cap was placed on the sample and the top of the cell secured to the lucite cylinder. The shaft was inserted through the top of the cell until it gently rested on the top cap.

The triaxial cell was then aligned under the upper head of a constant-strain load machine, a dial gage being attached to the upper head. Load was applied at a rate of 0.075 inch per minute of vertical head-travel. A load reading was taken and recorded for each 0.025 inch increment of strain.

Immediately following failure of the sample, the sample was removed from the cell and a moisture sample taken.

The compressive strength was taken as the average stress obtained from the failure-load readings of the two samples. Whenever the value of compressive strength for one sample varied from that of the other by more than 10 per cent, a new set of samples was molded and tested.

Test Results

Soils Combined With Stone Screenings and Cement

An evaluation was made of the effects of varying screenings contents and cement contents on each individual soil. Also, a comparison of the relative effects on the five different soils was made.

The influence of cement content on compressive strength, for a fixed screenings cement was found from the test data. This data indicates that, for any given curing period and lateral pressure, increasing the cement content caused an increase in the deviator stress.

For Soils IV, VII, and IX without screenings the greatest gains in strength, within any cement content range of 4 per cent, were obtained in the range from 0 to 4 per cent. For Soils V-A and VIII without screenings the largest per cent increases were found to be in the cement content range between 4 and 8 per cent. This information was derived from Tables XIII through XVII.

For soils containing 25 per cent screenings the same tables show that the highest strength gains for Soil IV were between 0 and 4 per cent. This range was also the most beneficial for Soils VII and IX. For Soil V-A, the greatest increase occurred between 4 and 8 per cent cement content. Soil VIII shows an increase in deviator stress that is constant throughout the entire cement content range.

The values of deviator stress for soils containing 50 per cent screenings indicate that the greatest increase in strength for all soils except Soil V-A occurred in the 0-4 per cent cement content range. Increasing the cement content from 4 to 8 per cent doubled the strength of Soil V-A giving that soil its largest increase for any 4 per cent range.

The influence of screenings content on the compressive strength of the different soils can be determined from Tables XIII through XVII. For all soil-cement combinations, the strength increased with increasing screenings content. However, the addition of screenings to soils not containing cement generally did not improve the strength; in fact, decreases in strength often occurred.

In order to determine the actual value of stone screenings as a substitute for portland cement in a stabilized base course, it is necessary to establish a design compressive strength. Since a minimum confined (20 psi) compressive strength of 300 pounds per square inch after 7 days curing is used by the Georgia State Highway Department for primary roads, this value was adopted for comparison purposes in this study.

The estimated percentages of cement necessary to produce a deviator stress of 300 psi are shown in Table XVIII. These values were obtained by interpolation from Figures derived from tables XIII through XVII. The relative effects of adding screenings to different soils having various percentages of cement are evident in these tables. Soil IV benefited from the addition of up to 50 per cent screenings for all four conditions of curing and confining pressure employed. For a 7-day confined strength of 300 psi the cement content was reduced from 6.4 per cent to 3.1 per cent by the addition of 25 per cent screenings.

For Soil V-A, the cement contents necessary to provide the design strength were lowered appreciably by the addition of 25 per cent and 50 per cent screenings. For a 7-day, confined strength of 300 psi the cement content was lowered from 6.4 to 3.7 by adding 50 per cent screenings.

The addition of screenings to Soil VII did not cause reductions in cement content under all conditions of curing and lateral pressure; in those instances in which reductions did take place the cement content was not lowered by more than 30 per cent. For the higher percentages of screenings content used the cement content was unaltered or even increased.

The most drastic reductions in cement contents occurred in Soil VIII where the cement percentage steadily diminished with increasing screenings content. Considering the 7-day confined values, the cement content declined

Table XVIII. Estimated Percentages of Cement Needed for a Compressive Strength of 300 lb./sq. in.

Soil No.	Screenings Content (%)	Cement Content (%)			
		Confined (20 psi)		Unconfined	
		7-day	28-day	7-day	28-day
IV	0	6.4	3.5	7.0	4.4
	25	3.1	2.4	3.5	2.9
	50	2.2	1.6	3.1	2.4
	75	2.2	1.2	4.1	2.9
V-A	0	6.4	5.3	7.6	5.5
	25	4.5	4.1	5.8	4.2
	50	3.7	3.0	4.1	3.5
	75	3.0	1.7	6.5	2.4
VII	0	3.1	2.9	3.3	2.7
	25	2.5	1.9	3.2	2.3
	50	2.6	1.7	4.0	2.3
	75	2.7	2.0	4.7	4.9
VIII	0	8.9	8.1	12 +	9.5
	25	6.5	6.1	8.4	6.8
	50	2.9	1.8	3.2	2.3
	75	1.7	0.9	2.4	1.2
IX	0	6.9	5.8	8.8	6.8
	25	4.2	2.3	5.0	3.5
	50	4.9	3.5	4.9	3.7
	75	6.9	4.5	12 +	6.0

from 8.9 per cent for the soil-cement mix (without screenings) to only 2.9 per cent with the addition of 50 per cent screenings; for 75 per cent screenings, only 1.7 per cent cement was needed to provide the design strength. It should also be noted that for the 28-day, unconfined compressive strength of 300 psi the cement content was reduced from 9.5 per cent to only 1.2 per cent.

The values given in Table XVIII for Soil IX produce a pattern somewhat like that of Soil VII; i.e., the only instances in which the cement contents were lowered appreciably were when screenings content was 25 per cent. However, it can be seen by comparing the data for these two soils that the reduction for Soil IX were considerably greater than those occurring in Soil VII. In fact, except for Soil IV, the addition of 25 per cent screenings were more beneficial to Soil IX in reducing cement content than it was to any other soil.

Although Table XVIII gives the percentages of cement needed to yield deviator stresses of 300 psi for screenings contents of 0, 25, 50 and 75 per cent it does not reveal the most economical cement percentages at intermediate percentages of screenings. From the standpoint of economy these values could be extremely important. The intermediate values can be determined by plotting the per cent stone screenings/percent cement ratios against the corresponding deviator stress values obtained from the strength tests, and connecting these points. Typical design curves are shown in Figures 18 through 21. The points were plotted from Tables XIII through XVI. For example, the deviator stress (7 days, confined) for Soil IV combined with 25 per cent screenings and 4 per cent cement is 429 psi; this value was plotted above the abscissa value 25/4, or 6.25. Likewise, the deviator stress of 418 psi for 75 per cent screenings and 4 per cent cement was plotted above the abscissa

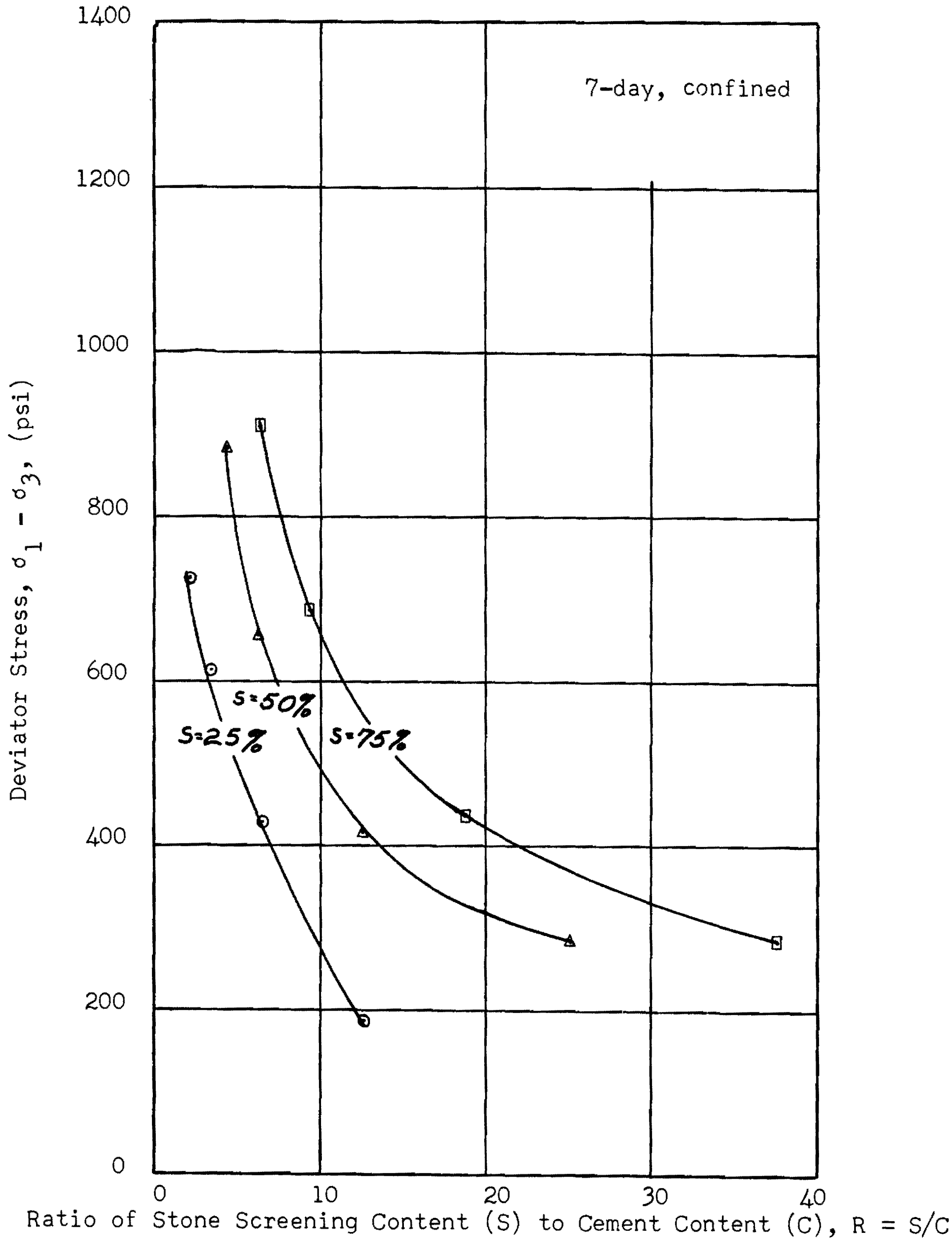

Figure 18. Design Curve, Soil IV Combined with Stone Screenings and Portland Cement.

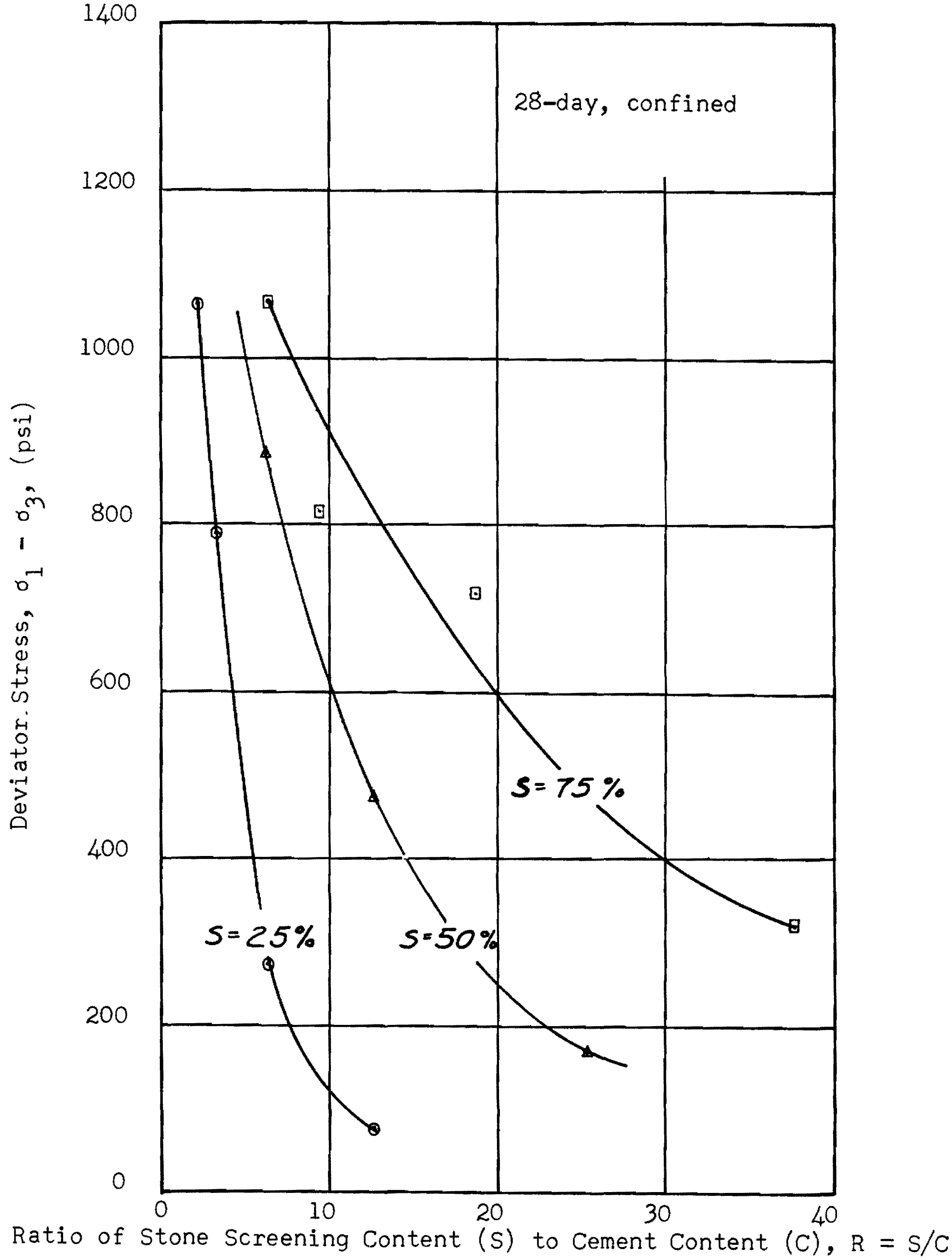

Figure 19. Design Curve, Soil V-A Combined with Stone Screenings and Portland Cement.

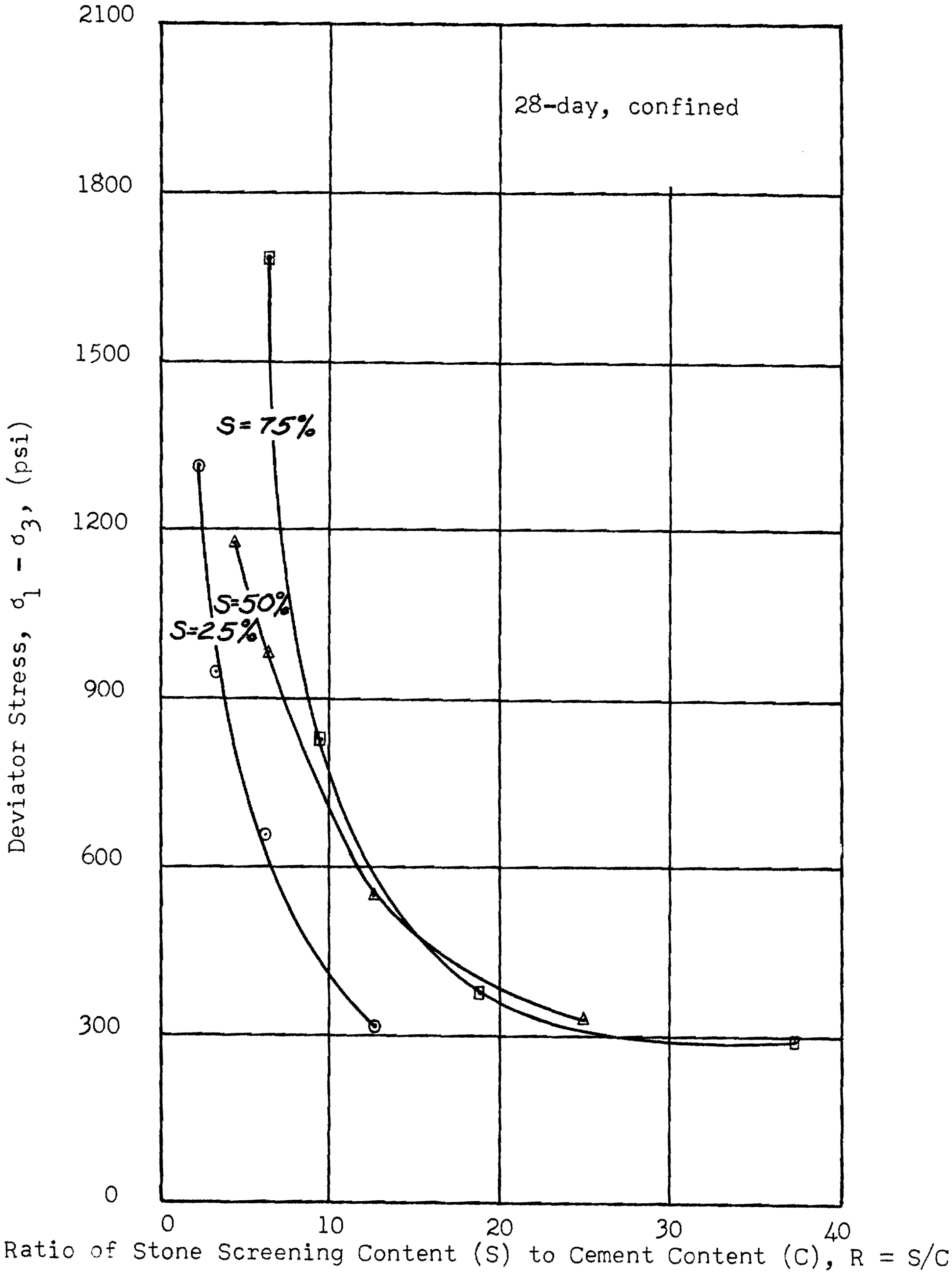

Figure 20. Design Curve, Soil VII Combined with Stone Screenings and Portland Cement.

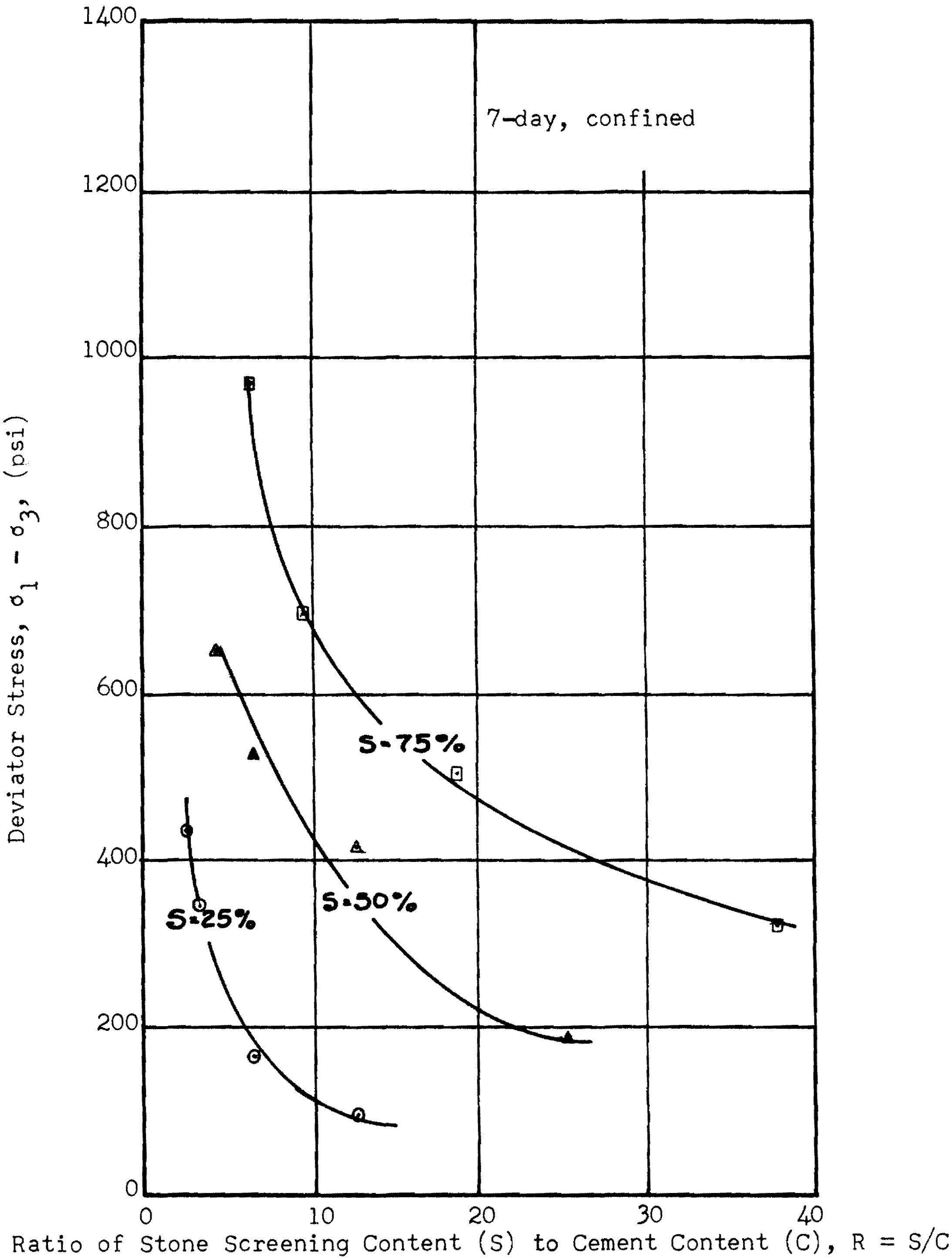

Figure 21. Design Curve, Soil VIII Combined with Stone Screenings and Portland Cement.

value 75/4, or 18.8.

It can be seen from Figures 18 through 21 that an infinite number of ratios of screenings content to cement content exist which theoretically give a compressive strength of 300 psi for any given curing period and confining pressure. However, with a little experience, one can quickly determine the best combinations for design -- assuming, of course, that the costs of cement and screenings are known. The most economical combination of Soil V-A, screenings, and cement having a deviator stress of 300 psi may be found by first drawing a horizontal line at 300 psi. Then, by picking several values of "R" along this line, the best combination can be determined. For instance, at an "R" value of 10, the screenings content (S) is approximately 35 per cent; therefore, the cement content (C) is 3.5 per cent.

Conclusions

Several significant conclusions were drawn from the results obtained from these tests. These are as follows:

1. Almost all of the gain in dry density caused by the addition of portland cement and stone screenings to a soil was due to the stone screenings.
2. For any fixed screenings content, increasing the cement content produced an increase in the compressive strength.
3. In all soils tested, the addition of stone screenings was found to reduce the amount of cement required to develop the design compressive strength. However, adding only stone screenings to the soil did not cause the compressive strength to be increased.
4. All soils combined with stone screenings, except Soil V-A, showed more of a strength increase in the 0-4 per cent cement content range than in any other 4 per cent range.

5. The compressive strength of a soil containing cement and stone screenings did not increase with increased density.
6. The compressive strength of Soil VIII, considered to be the poorest soil according to the AASHO Soil Classification System, was improved much more than any other soil by the addition of stone screenings.

CHAPTER V

PHOSPHORIC ACID AND LIME-FLYASH

Phosphoric acid is a relatively new product in the field of stabilization but it has shown some stabilizing qualities in experimental work.[12] Compacted plastic clay soils with about 2 per cent phosphoric acid content showed greatly improved resistance to water and weathering, but the mechanism of soil-phosphoric acid stabilization was not explained. From agriculture it was found that phosphates are fixed in soil.[13] Sodium phosphates are sometimes used to disperse soils in water for particle size analysis.

Previous work indicates that phosphoric acid, used in amounts from 1 to 10 per cent of the dry soil weight, was an economical and promising stabilizer for fine grained soils.[14] Strengths depended on water content and density; greater strengths were found when specimens were cured under humid conditions; and certain additives accelerated the curing process and improved the strength retention after immersion. From more recent studies it was concluded that phosphoric acid with added fluorine compounds and/or amines promised low-cost stabilization of fine grained, carbonate-free soils under field conditions.[15, 16]

Lyons[12] described some work with phosphoric acid as a stabilizer. This work showed that soil stabilized with about 2 per cent phosphoric acid became less plastic, was easier to mix and increased the strength of the mix.

The combination of lime and flyash used as an admixture in soil stabilization is fairly new. A patent was granted in 1954 on the use of lime and flyash combinations for the stabilization of soils.[17]

The lime-flyash combination has been used on stretches of primary and secondary roads in Maryland, shoulders and interchanges on the New Jersey

Turnpike, and airport runways in Pennsylvania, New Jersey, and Missouri.[18,19,20]

Using lime-flyash for stabilization bases has proven in many cases advantageous. The cost is said to be one-half that of most bases; it produces high thermal resistance; and due to its slow rate of pozzolanic reaction, allows construction to be interrupted for extended periods without harm to the final product.[21]

Materials Used and Test Methods

General

The objective of this experiment was to determine how certain percentages and ratios of phosphoric acid and lime-flyash combined with different Georgia soils compare with other admixtures added to the same soil in relatively the same proportions.

It was not the purpose of this experiment to delve deeply into chemical make-up of these admixtures and explore all the possibilities of using them as stabilizing agents.

Soils Used

The soils used in this experiment were Soil I, II, III, IV and V. The classifications of these soils are shown in Table I.

Lime-Flyash

The lime-flyash combination was added to the soil at a constant proportion of 25 per cent of the dry weight of the soil. The only variable was the ratio of the lime to the flyash. The lime to flyash ratios used were 1:1, 1:2, 1:5 and 1:9. The lime used in the mixture of lime and flyash was a hydrated high calcium lime purchased on the open market. Analysis of the flyash is shown in Table XIX.

Table XIX. Flyash Analysis

	Macon, Ga.	Columbia, S. C.
Chemical composition, %		
Silicon dioxide, SiO_2	41.40	45.92
Aluminum oxide, Al_2O_3	21.05	32.00
Ferric oxide, Fe_2O_3	8.65	16.50
Magnesium oxide, MgO	5.36	1.40
Sulphur trioxide, SO_3	1.16	0.84
Carbon, C	1.66	2.32
Loss on ignition	3.12	2.24
Specific surface area, Blaine (sq. cm/gm)	3427	1760

Phosphoric Acid

The phosphoric acid used was a 85 per cent solution and was added to the soil by per cent of dry weight of the soil. The percentages used were 3, 5, and 7. The 85 per cent solution was accomplished by combining the acid and water and then mixing for ten minutes.

Testing Procedures

The testing procedures for the phosphoric acid and lime-flyash additives were similar to those in Chapters II, III and IV.

Moisture-Density Tests

Moisture-density relationships were determined for each of the five soils, Soil I, II, III, IV and V, using the two additives with their respective ratios and percentages. These results are given in Tables XX and XXI. A small increase in the density of all five soils was noted with the addition of phosphoric acid with no significant change in optimum moisture content. Adding lime and flyash produced a decrease in density and an increase in optimum moisture in all the soils except Soil V. Adding lime and flyash to this soil caused an increase in density and optimum moisture.

Test Results

In evaluating the molded samples, only samples molded within 1 per cent of optimum moisture were used. For each test, 4 samples were molded for unconfined compression at 7 and 28 days and 4 samples for triaxial testing at 7 and 28 days. For compressive strength evaluation, only the values which were within 10 per cent of the average of the other samples were used. In most instances, the results were consistent and represent the average of 4 samples tested. Compressive strength results are shown in Tables XXII and XXIII. Figures 22 through 24 show curves of compressive strengths versus various admixtures for the confined triaxial tests.

Table XX. Maximum Dry Density and Optimum Moisture Using Lime-Flyash Combinations

Lime-Flyash Ratio	Soil I		Soil II		Soil III		Soil IV		Soil V	
	γ_d	ω	γ_d	ω	γ_d	ω	γ_d	ω	γ_d	ω
1:1	114.3	13.5	118.7	10.8	114.4	11.0	101.0	20.0	101.5	20.2
1:2	112.1	14.0	119.8	10.3	112.8	11.5	102.1	19.8	102.0	20.0
1:5	111.0	13.7	120.5	10.2	109.1	12.1	102.0	20.0	101.3	19.8
1:9	108.6	14.0	120.8	10.4	108.2	13.0	102.0	20.0	101.4	19.7

γ_d is maximum dry density (lbs/cu. ft.)

ω is the optimum moisture content (%)

Table XXI. Maximum Dry Density and Optimum Moisture Using Phosphoric Acid

Acid %	Soil I		Soil II		Soil III		Soil IV		Soil V	
	γ_d	ω	γ_d	ω	γ_d	ω	γ_d	ω	γ_d	ω
1	124.2	9.3	124.9	10.5	---	---	115.8	15.2	115.8	15.9
2	125.4	9.0	125.6	9.4	---	---	117.2	14.5	117.4	14.7

γ_d Maximum Dry density (lbs/cu.ft.)

ω Optimum moisture content (%)

Table XXII. Average Results of Strength Test for Lime-Flyash Combination Additive (Axial Stress, σ , Psi)

Lime-Flyash Ratio	Soil I				Soil II				Soil III				Soil IV				Soil V			
	7 Day		28 Day		7 Day		28 Day		7 Day		28 Day		7 Day		28 Day		7 Day		28 Day	
	0*	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20
1:1	48	133	57	159	128	232	153	277	21	142	14+	106+	53	128	65	149	48	130	113	199
1:2	41	138	36	115	115	229	140	266	7	108	12+	62+	45	116	51	122	44	126	91	177
1:5	58	156	74	180	88	197	100	224	9	94	11+	89+	43	108	51	143	34	106	75	152
1:9	16	96	25	110	72	191	85	205	8+	70+	8+	78+	42	108	49	128	37	100	65	143

*NOTE: The 0 and 20 indicate confining pressure in psi in triaxial test.

\+ These samples were molded with Columbia, S.C. flyash.

Table XXIII. Average Results of Strength Tests for Phosphoric Acid Additive (Axial Stress, σ , psi)

Acid	Soil I				Soil II				Soil III				Soil IV				Soil V			
%	7 Day		28 Day		7 Day		28 Day		7 Day		28 Day		7 Day		28 Day		7 Day		28 Day	
	0*	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20	0	20
1	14	126	27	133	34	126	41	135	2	70	2	70	41	103	46	117	46	77	63	108
2	18	95	29	120	34	120	47	143	1	68	2	70	78	149	115	182	58	100	82	129

*NOTE: The 0 and 20 indicate confining pressure in psi in the triaxial test.

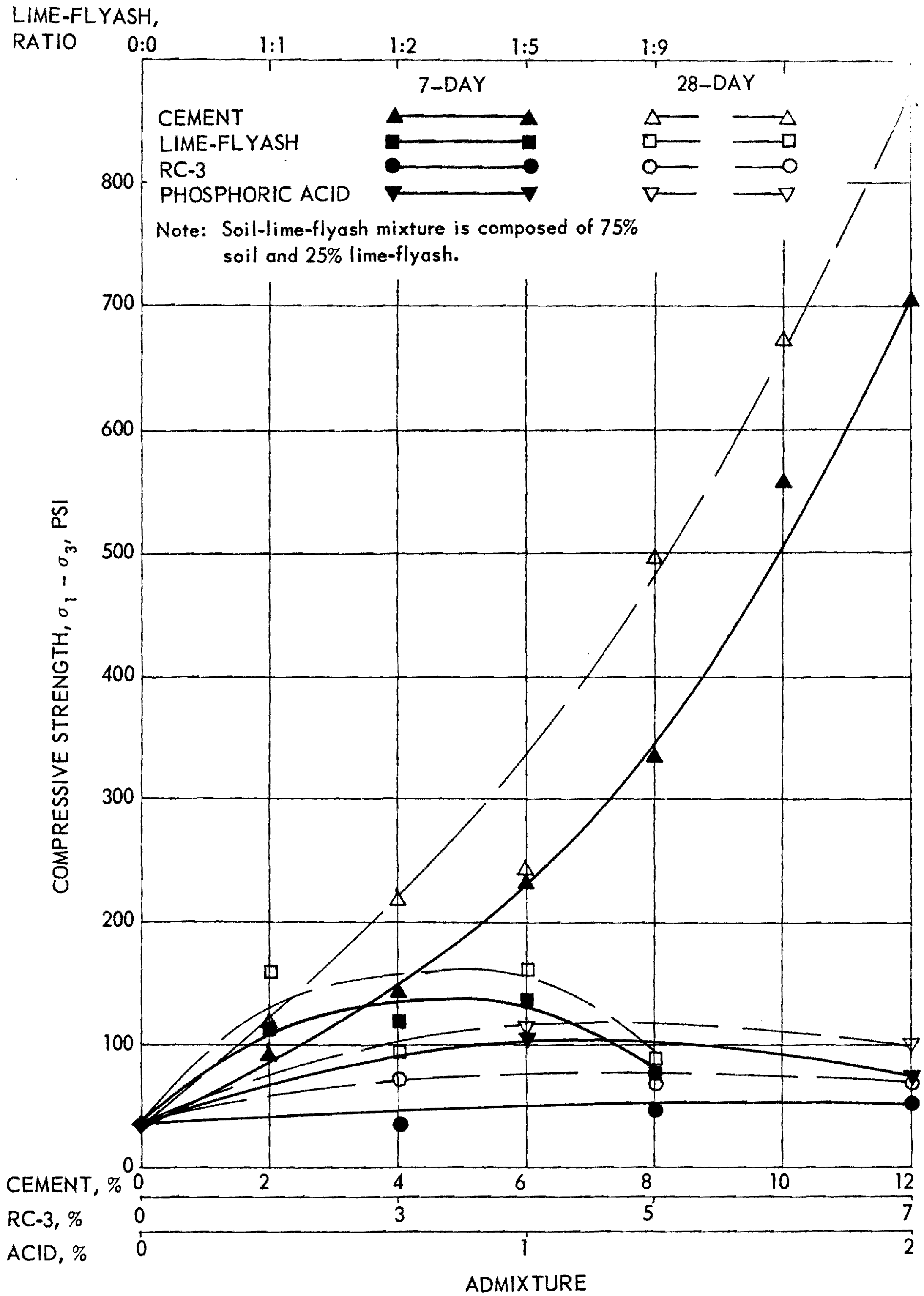

Figure 22. Relationship of Confined Comp·essive Strength and Admixture for Soil I.

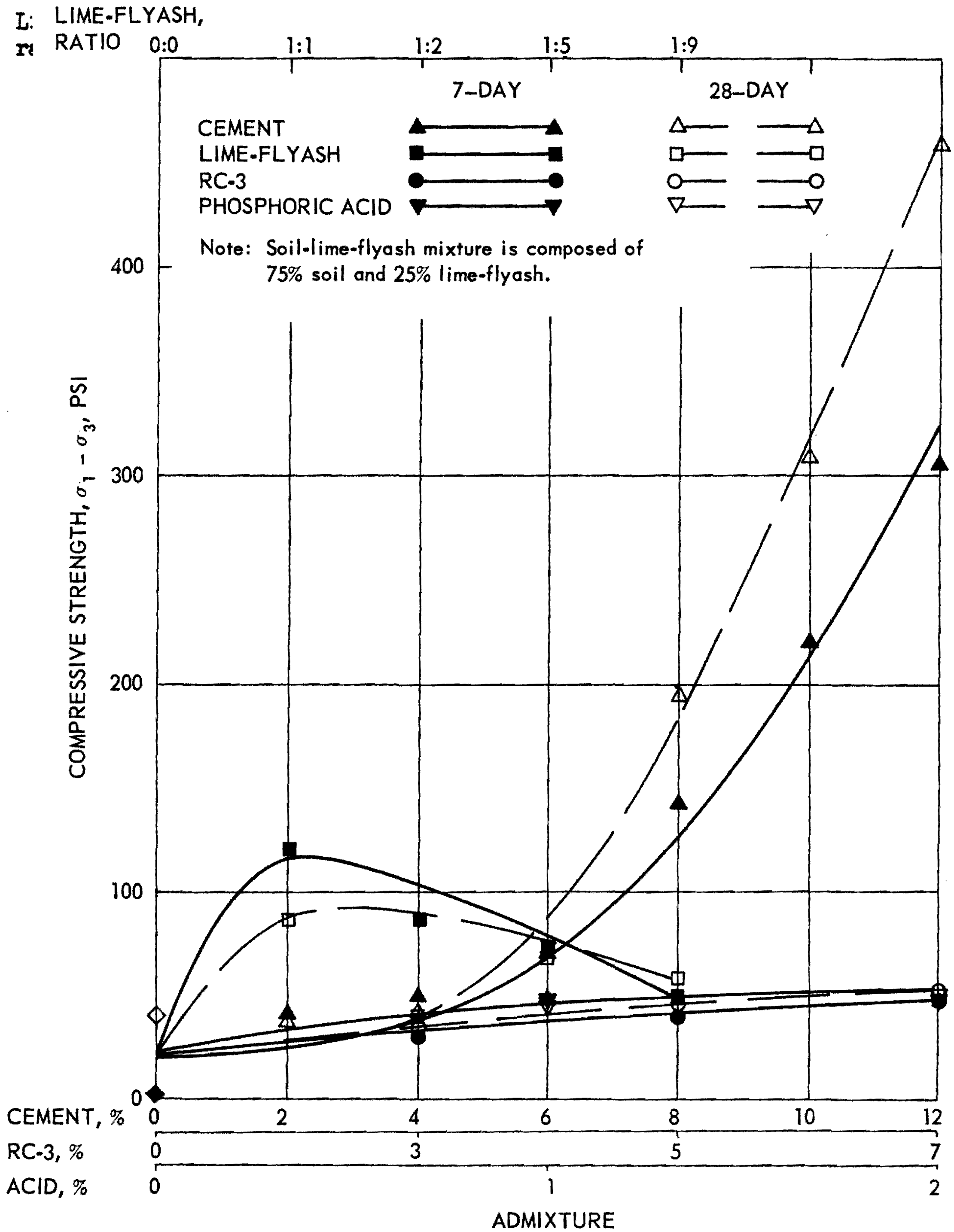

Figure 23. Relationship of Confined Compressive Strength and Admixture for Soil III.

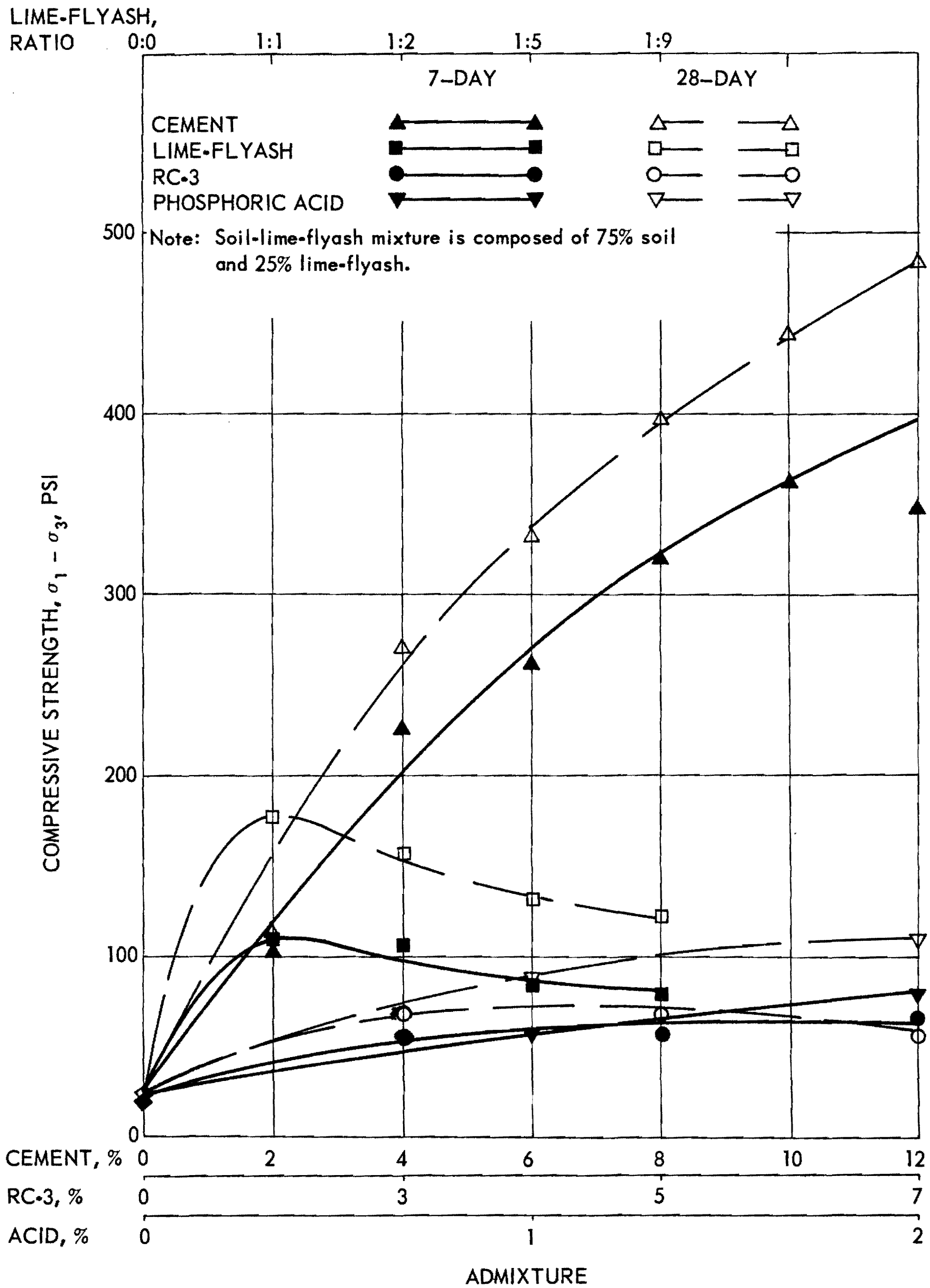

Figure 24. Relationship of Confined Compressive Strength and Admixture for Soil V.

Conclusions

The addition of 25 per cent lime-flyash improved the strength of all soils. A 1:1 lime-flyash ratio gave the greatest strength improvement except Soil I, where little change was noted from a 1:1 to a 1:5 ratio. All of the lime-flyash soil mixtures increased in strength with increased curing time.

Phosphoric acid caused a nominal increase in strength of all soils. The greatest improvement with this admixture was in the finer grain soils with the higher clay content. The two per cent acid content gave a greater strength increase than did the 1 per cent. The strength after curing for 28 days was higher than after 7 days curing in all soils except Soil III.

In view of the amount of research done by other investigators with the use of these admixtures, it would be highly desirable to continue the work with these admixtures with Georgia soils.

CHAPTER VI

CRACKING IN CEMENT TREATED BASES*

Concurrently with the development of soil-cement bases some technical problems developed. The major technical problem was the occurrences of large shrinkage cracks during the hardening process. The correction of this problem, to a great extent, was found by cutting the cement-content ratio in half so as to have present in the soil three to four per cent cement content. The resulting cement treated base was weaker and developed more shrinkage cracks at closer intervals, but the individual cracks were smaller and could be corrected more effectively with some sort of bituminous mix sealing operation.[22]

The occurrence of shrinkage cracks are caused by other than cement content ratio. Some of these causes are attributive to the amount of clay content present in the soil and improper curing.

The amount of shrinkage increases concurrently with the clay content. A majority of these cracks extend into the soil-cement only one to three inches with full width being five feet or more apart.[23] These cracks can be appreciably reduced by the addition of flyash. From this addition though, there would be a definite decrease in strength depending on soil texture.[24]

Soil texture has a definite effect on the benefits of using flyash as an additive in soil-cement. There is a compressive strength loss in plastic loess (41.8% 5 microns clay) and alluvial clay (74.3% 5 microns clay) but a reduction in shrinkage cracking. The best strength gains due to pozzolanic reaction appear in sands, due probably because of its low clay content.[24]

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*The work on Cracking in Cement Treated Bases was initially a part of the research program. This pilot study was made before the project was terminated and is reported herein.

It can be concluded from the above that if there existed in the soil-cement a high sand content and low clay content, there would not exist sufficient justification for the addition of flyash for shrinkage reduction purposes. Sufficient justification would exist though for the using of flyash for the economical purpose of reducing the cement content.

The type of soil is the most important single factor affecting the quality of soil-cement. If the soil is unsuitable, little can be done, at the present time to make the resulting soil-cement satisfactory. In general, experience has shown that soils meeting the following conditions can be hardened effectively through the addition of reasonable amounts of cement:[25,26]

Per cent finer than 0.002 mm less than 35%

Per cent passing No. 4 seive (4.76mm) greater than 55%

Maximum size equals 3 in.

Liquid limit less than 50%

Plasticity index less than 25%

Investigations by Maclean[27] have indicated that the nature of the action associated with the clay, as well as the type of clay mineral, influences the response of a soil to cement stabilization. He found that calcium clays were the most easily stabilized, whereas sodium and hydrogen clays were more difficult. The addition of hydrated lime to sodium and hydrogen clays in order to convert them to the calcium form has resulted in satisfactory soil-cement. Experience has shown that soils composed of the non-expansive clay minerals are more suitable for cement stabilization than soils composed of the expanding lattice-types minerals.

A knowledge of pedological soil classification systems is helpful when considering soil-cement stabilization. Soils of the same texture, horizon, and series have been found by the Portland Cement Association to require about the same cement treatment.[28]

A detailed study of the effect of organic matter on soil-cement was undertaken by Clare and Sherwood.[29] They found that organic compounds with high molecular weights, such as cellulose, starch, and lignin, did not affect strength; while those of lower molecular weights, such as nucleic acid and dextrose, acted as hydration retarders and resulted in lower strengths.[30]

Improper curing is a problem created largely by negligence. If the design water content is not properly retained after placement, then there will exist improper hydration. There are numerous methods of curing which have been proven effective.

Although the large shrinkage cracks can be controlled to a great extent, the direct effects from the smaller cracks and weakened base on the subgrade have not been determined. The effects have been determined to some extent on the surface.

Actual experiments have shown that when the soil-cement cracks after the bituminous surface has been placed the effects on the surface is small and will most probably be sealed by the traffic. Also, if cracks occur before the placement of the bituminous surface, which is normally the case, the bituminous mix itself will fill and seal the cracks.[23] The sealing of these cracks has a tremendous influence on the service life of the soil-cement because this allows prevention of water filtration. Water permitted to filter down along the cracks will not materially weaken the soil-cement itself, however, if the cracks protrude completely through the soil-cement base then water will be able to filter to the subgrade and consequently will dampen and weaken the subgrade.[31]

Scope of Experiment

The objective of this research is to determine how certain factors affect cracking in soil-cement with emphasis being placed on the cracking that occurs at an early age (during the curing period).

For the purpose of experimentation, some rational assumptions were made. The first assumption was that most cracking in soil-cement mixtures can be attributed to the clay that is present in the soils, consequently, predominantly clay soils were specified. Other assumptions were that moisture content, cement content, and temperature differential (temperature gradient across sample) are factors in cracking. It was also assumed that in soil-cement bound macadam the aggregate would not cause cracking.

From these assumptions, three different tests were devised. These tests were designated as Phase I, Phase 2 and Phase 3.

Description of Materials and Testing Equipment

Soils

A total of four different soils were used in this experiment. These soils were designated as Soil A, Soil B, Soil C and Soil D.

Two of these soils, Soil B and Soil C were anticipated in the field predominately clay while Soil D was the optimum soil used in soil-cement bound macadam in the state of Georgia.

Soil A, a well graded, dark red, sandy loam soil, was obtained from Bartow County. Soil A was used in the cement stabilized base on U. S. 41 between Rome and Cartersville, Georgia.

Soil B, a well graded, brownish yellow, sandy clay loam soil, was also obtained from Bartow County. This soil was used as the sub-base for the same project as Soil A.

Soil C is a well graded, light red, sandy loam soil obtained from Fulton County.

Soil D is a well graded, brownish gray, sandy loam soil obtained from

Douglas County.

Physical Tests

After securing the soils, only those portions passing a No. 4 U. S. Standard Sieve were placed in containers for use in the experiment.

The following standard tests were performed on each of the four soils for identification and classification:

1. Grain size and analysis as specified in AASHO Designation T 88-57.
2. Plastic limit as specified in AASHO Designation T 90-56.
3. Liquid limit as specified in AASHO Designation T 89-60.
4. Specific gravity as specified by AASHO Designation T 180-57.
5. Volume change as specified by G.H.D. - 800.09.

Tabular results of these physical tests are given in Table I.

Admixtures

The only admixture used in this experiment was Type I Portland Cement. The chemical composition of this admixture is given in Table XXIV.

Test Equipment

Mixing Equipment. The soil, cement, and water needed for the 6"x6"x18" samples were blended with a Read Standard Grant mixer equipped with a hook blade. The mixture was blended at a speed of 125 revolutions per minute. Mixer is shown in Figure 25.

The mixture needed for moisture density samples was blended with a Hobart C-100 mixer equipped with a flat blade. Mixing speed was 144 revolutions per minute.

Compaction Equipment. The 6"x6"x18" samples were compacted with a modified Rainhart mechanical compactor equipped with a 11 pound 1"x5-7/8" rectangular faced hammer.

This compactor was calibrated to the Standard Proctor. The number of

Table XXIV. Portland Cement Analysis

Chemical Composition, %	
Silicon dioxide, SiO_2	20.46
Ferric oxide, Fe_2O_3	2.44
Aluminum oxide, Al_2O_3	5.90
Sulphur trioxide, SO_3	2.08
Calcium oxide, CaO	62.87
Magnesium oxide, MgO	4.18
Insoluble residue	0.30
Loss on ignition	1.38
Specific surface area, Blaine (sq. cm/gm)	3464

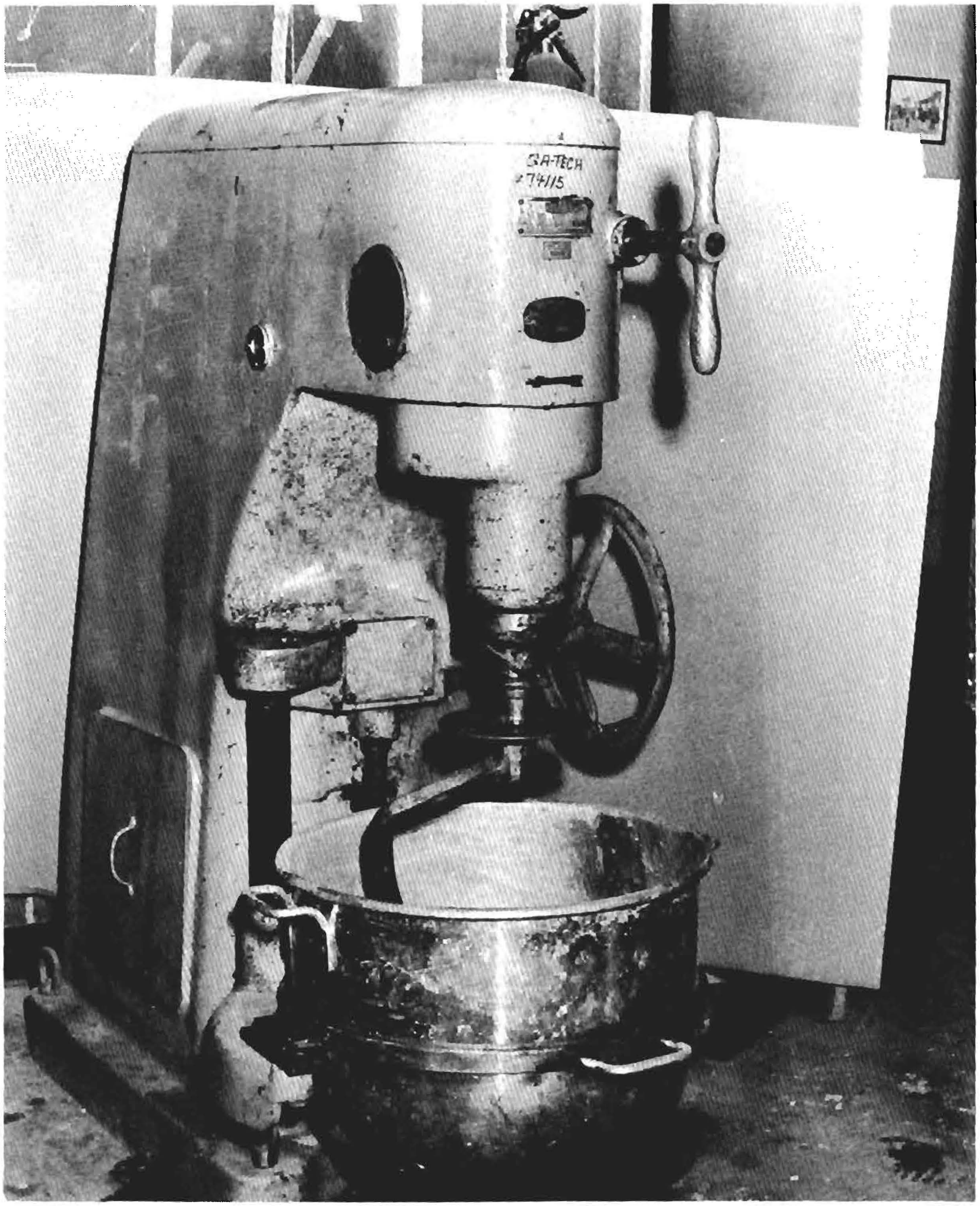

Figure 25. Read Standard Grant Mixer.

blows required per layer was first computed from energy equations with the resultant number being 125 blows. After compacting several samples, it was found that 123 blows per layer was correct. The soil used for this calibration was Soil D. Each of three 2" layers of the sample received 123 blows from a height of 12" above the surface of the soil. This compactor is shown in Figure 26. A detailed drawing of the mold is presented in Figure 27.

The moisture density samples were compacted with equipment as required for the Standard Proctor compaction test, AASHO designation, T 99-57.

Temperature Differential Equipment. Samples to be subjected to a temperature differential were placed in the apparatus, shown in Figure 28 and 29. Eight 250 watt bulbs maintained the top half of the sample at 140°F, while water circulating through sheet metal forms maintained the lower half at approximately 78°F.

Pilot Studies

The purpose of these pilot studies was to devise a testing procedure that would result in exploring only the significant factors related to cracking. Also, these studies would help in developing a systematical testing program.

Water Retention and Autogenous Shrinkage Properties of Type I Portland Cement

A thorough study was made on the water retention properties and autogeneous volume change in the paste of Type I Portland Cement. This study was performed in the following manner:

1. Hold the cement constant at 200 grams and vary the water from 50 grams in increments of 10 grams to 200 grams, a total of 16 different combinations being obtained.

2. Vigorously mix each sample 90 sec., then after mixing,

3. Seal the container to prevent any water from evaporating.

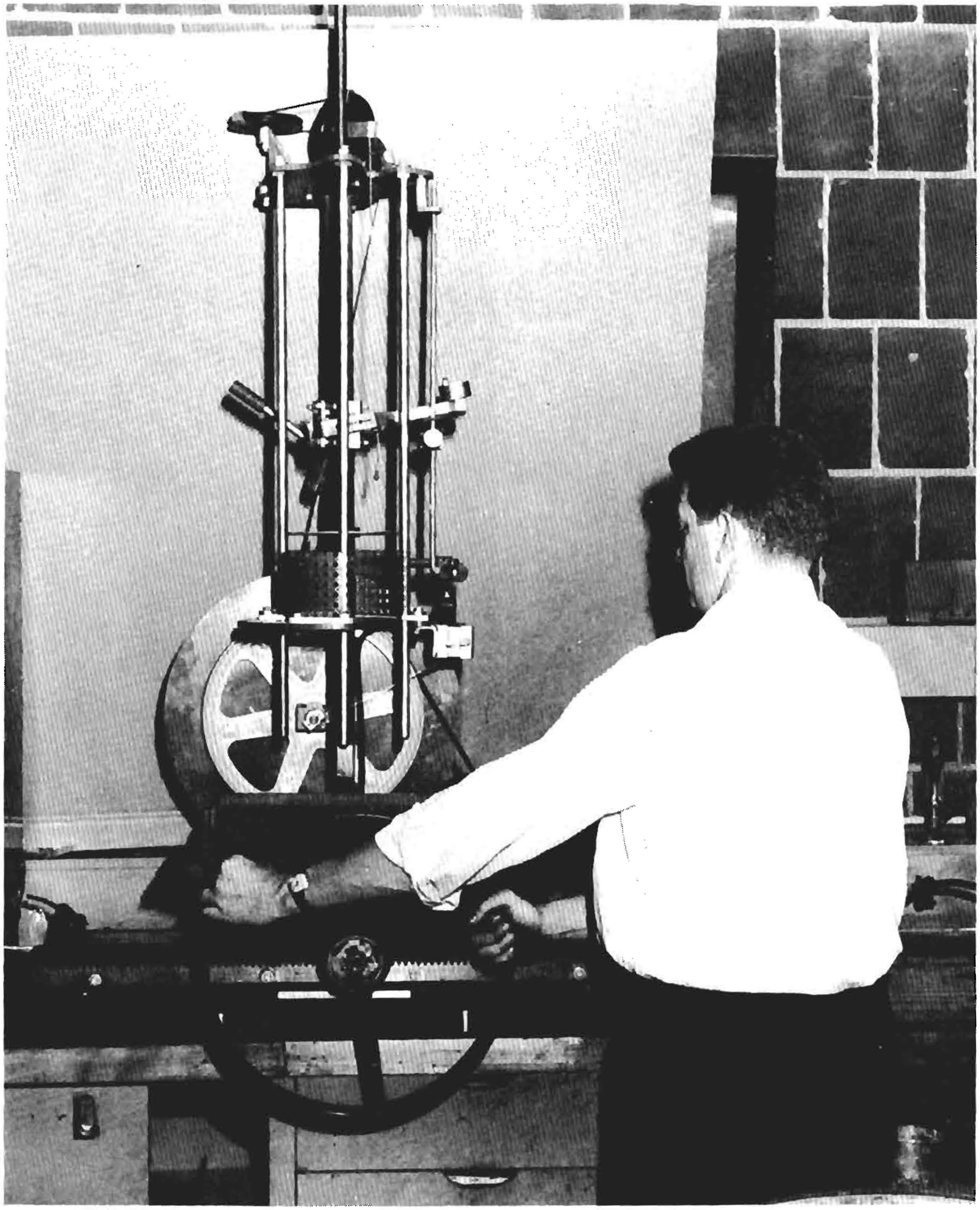

Figure 26. Modified Rainhart Mechanical Compactor.

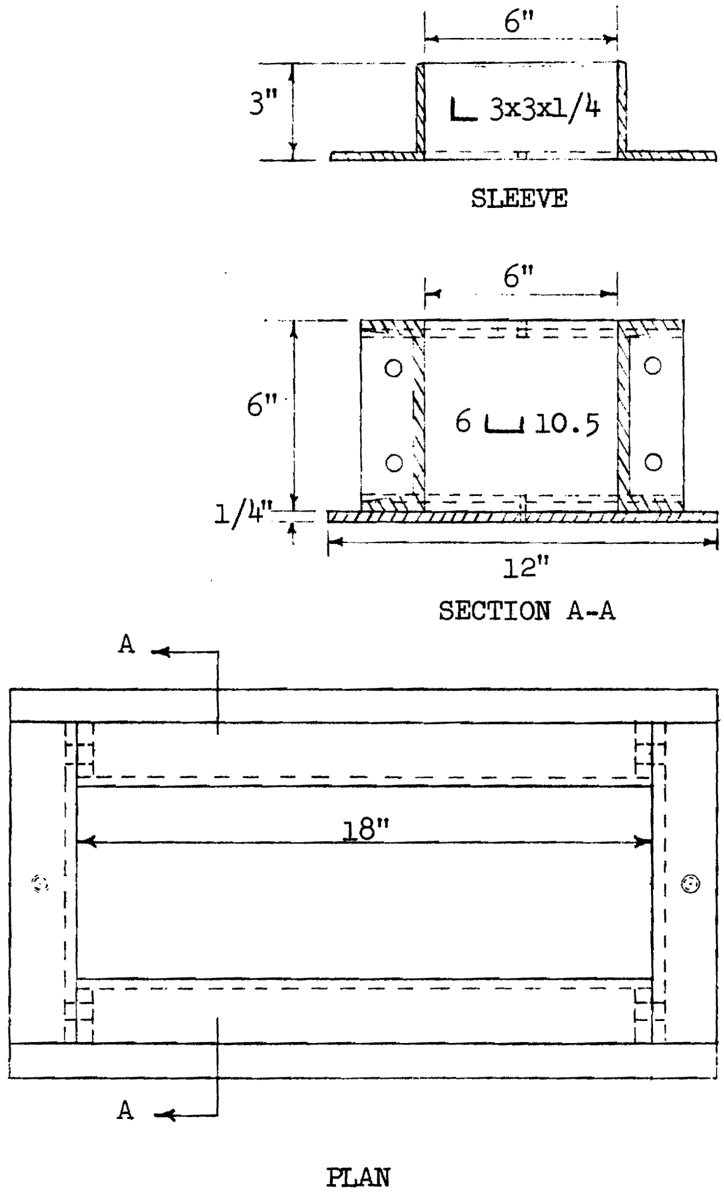

Figure 27. Test Specimen Mold.

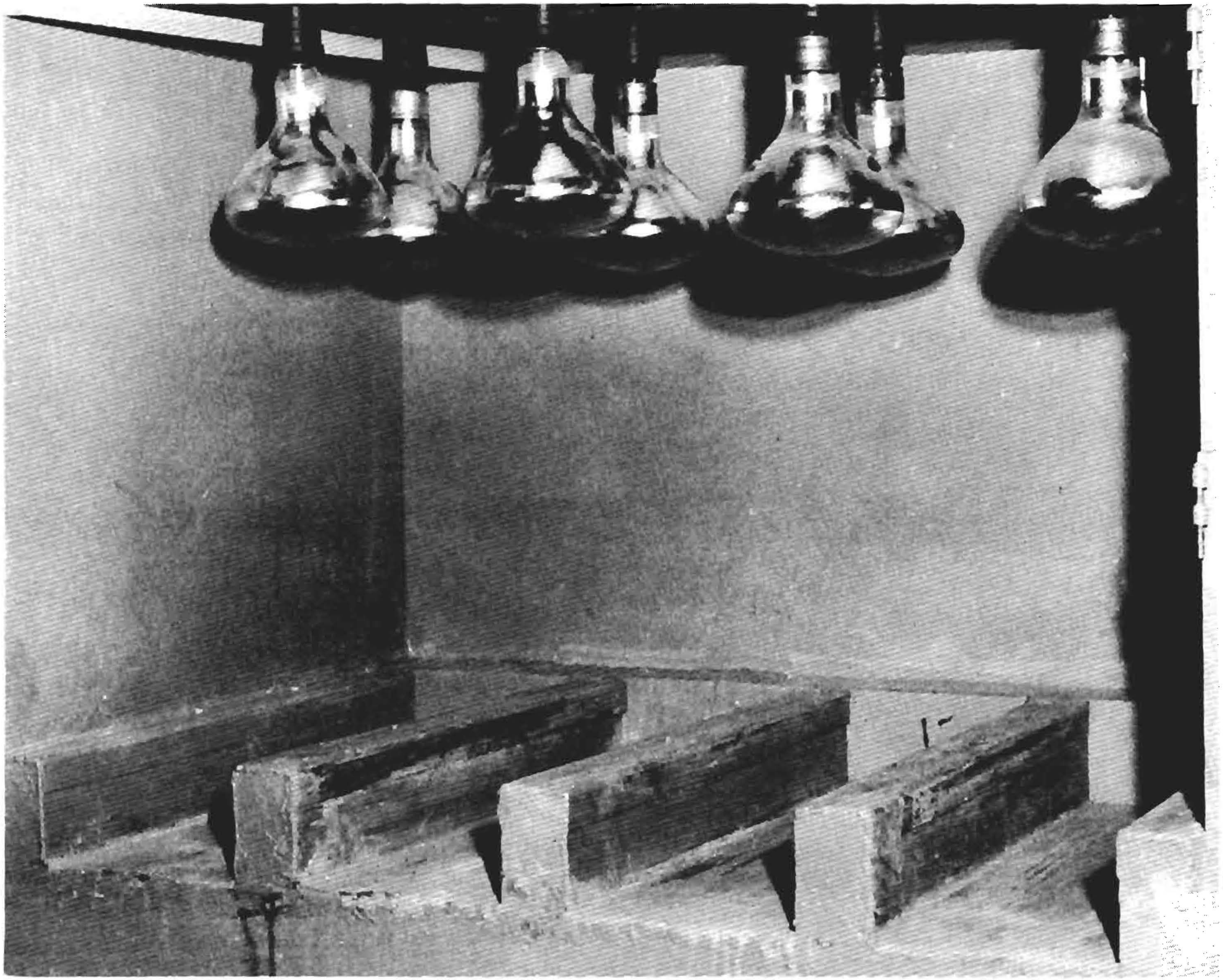

Figure 28. Temperature Differential Apparatus.

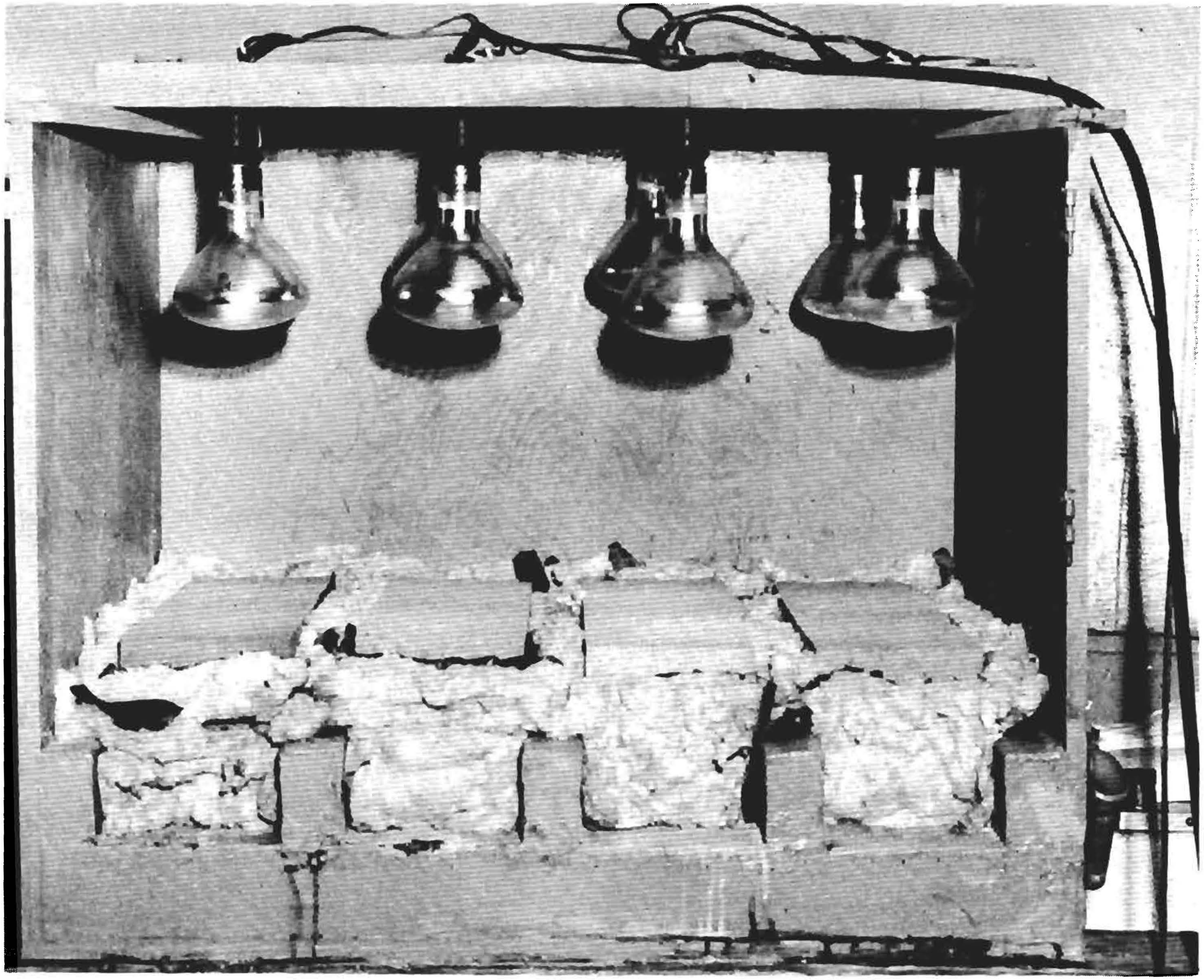

Figure 29. Temperature Differential Apparatus with Test Specimens.

4. Allow to cure at constant temperature.

5. Weigh each sample at 7, 28, and 60 days and record weights.

6. After a 60 day period, remove seals, place in oven at 110°C for a period of 7 days and record final weights.

It was found that the autogenous volume change during the 60 day period was about 0.25% of the weight of the cement. These values are shown in Table XXV. An interesting point to note here is that there was not any significant difference in volume loss between the different water cement ratios.

In determining the water retention properties or the Type I Portland Cement, it was found that the low water-cement ratios retained little water, but as the water-cement ratio increased the water retention also increased. This increase in water retention continued until the water cement ratio was about 0.5. At this point the water retention, which was about 36 grams, remained constant for the remaining larger water-cement ratios. Those values are given in Table XXV and graphs plotted from these values are shown in Figure 30 and 31.

Volume Change

A study was made of Type I Portland Cement and of the four soils, Soil A, Soil B, Soil C and Soil D for volume change characteristics due to the loss of evaporable water from the specimens. This volume change was determined by the Georgia State Highway volume change method, No. 800.09. This method entailed making two identical specimens of each soil, placing one specimen in a water bath and the other in an oven at 110°C for a 24 hour period. After this time interval the total amount of difference between the expansion and shrinkage of the respective specimens was recorded as per cent volume change with respect to the initial volume. Apparatus used for this test is shown in Figure 32.

Table XXV. Water Retention Properties of Type I Portland Cement

Can No.	Initial Wt. of Cement $+H_2O$	Water-Cement Ratio-(W/C)	Percent Wt. Loss After 60 Day Cure	Total Wt. of Water Retained After Entire Test (gm)	Percent Water Retained After Entire Test (gm)
1	250	0.25	0.40	23.5	47.0
2	260	0.30	0.38	27.6	46.0
3	270	0.35	0.26	30.2	43.1
4	280	0.40	0.29	32.9	41.1
5	290	0.45	0.34	33.1	36.8
6	300	0.50	0.23	34.7	34.7
7	310	0.55	0.26	34.9	31.7
8	320	0.60	0.25	35.4	29.5
9	330	0.65	0.24	37.0	28.5
10	340	0.70	0.24	35.7	25.5
11	350	0.75	0.20	34.6	23.1
12	360	0.80	0.30	36.6	22.9
13	370	0.85	0.24	36.7	21.6
14	380	0.90	0.21	36.1	20.1
15	390	0.95	0.15	36.5	19.2
16	400	1.00	0.25	36.3	18.2

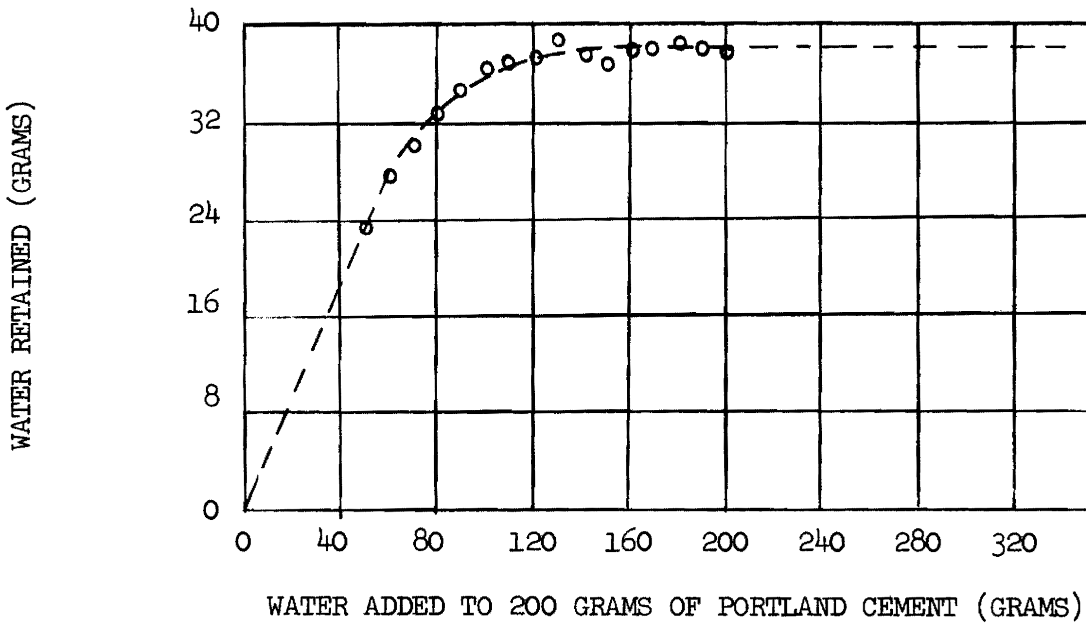

Figure 30. Water Retained Vs. Water Added to 200 Grams of Portland Cement.

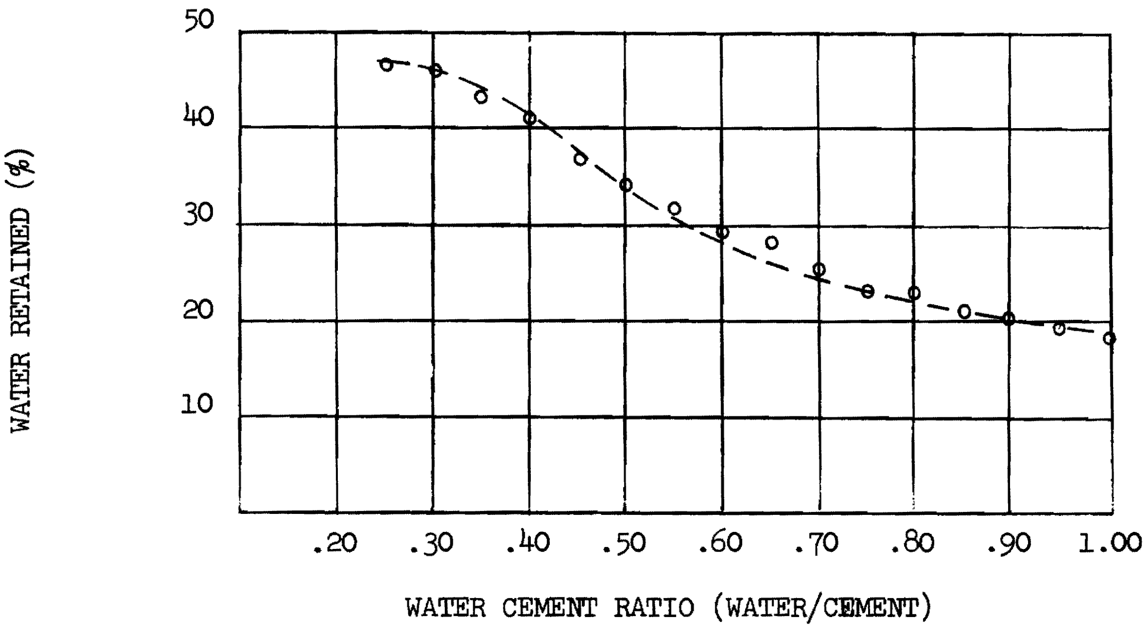

Figure 31. Per Cent Water Retained Vs. Water-Cement Ratio.

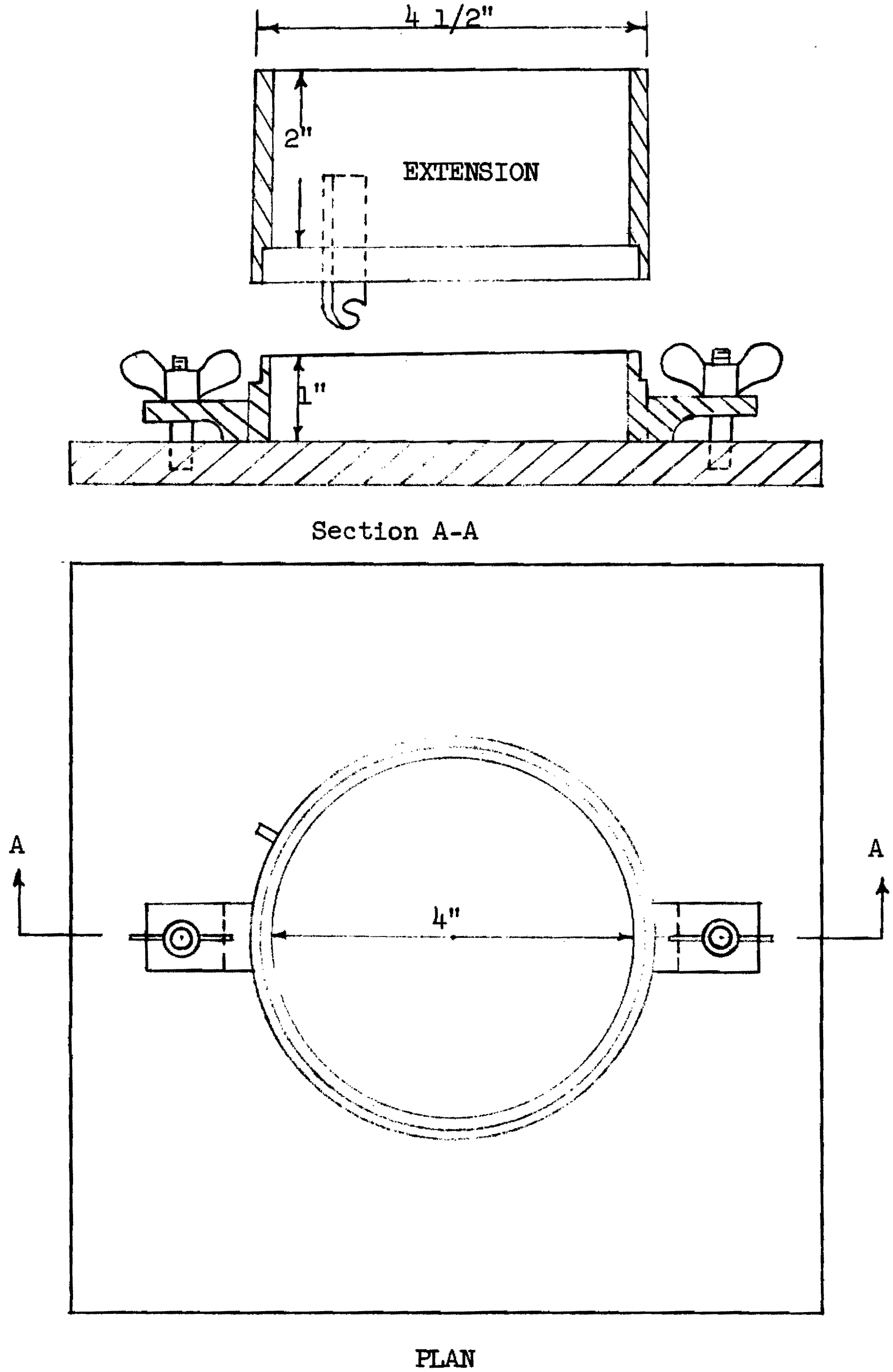

Figure 32. Volume Change Apparatus.

Due to the small size of this specimen used, the volume change recorded for the Type I cement was not large enough to be significant. The volume changes recorded for the four soils are shown in Table I.

Correlation of the "Speedy" Moisture Determination Method

This study was made to determine the correlation of the "speedy" moisture tester, manufactured by the Alpha Lux Company, Inc., to the actual moisture content by the oven dry method, for the four soils used in this experiment. Calibration curves were plotted from this data in Table XXVI. This calibration permitted more accurate hygroscopic moisture determinations to be made for the undried soils.

Sample Size and Mold Make-Up

An attempt was made in this study to develop a sample size that would be large enough to show any evidence of cracking and also be easily handled in the laboratory. It was also the purpose of this study to determine the best design and material for the mold to be used for making samples.

The resultant size arrived at was a rectangular mold 6"x6"x18". This mold is constructed of 6" channel iron with a 3"x3" angle iron being used for the sleeve. This mold is shown in Figure 27.

Relationship of Optimum Moisture Content to Various Cement Contents

This study was made to determine the relation of optimum moisture content to various cement contents for the different soils, i.e., what effect the different cement contents would have on the optimum moisture and density of the specimens. Moisture density relationship as designated by AASHO were run on Soils A, B, C and each having an 11% and 22% cement content. It was found that only one soil, Soil C, showed any significant change. The reason for this change is attributed to the light weight of Soil C. Results of this test are shown in Figures 33, 34, 35, 36.

Table XXVI. "Speedy" Moisture Determination Correlation With Soils A, B, C and D

Soil "A"		Soil "B"		Soil "C"		Soil "D"	
"Speedy"	Actual	"Speedy"	Actual	"Speedy"	Actual	"Speedy"	Actual
0.8	1.3	0.6	1.0	6.2	7.2	6.0	6.6
0.8	1.4	0.8	1.0	5.5	6.5	6.8	7.0
0.8	1.3	0.6	1.0	5.8	6.8	7.6	7.8
3.0	4.0	0.6	1.1	6.0	6.5	8.2	8.1
4.1	5.1	0.6	1.1	4.8	5.8	6.0	6.6
1.2	1.7	1.0	1.4	4.9	5.7	1.3	1.8
4.5	5.2	1.0	1.4	4.8	6.4	1.8	2.2
5.3	6.1	1.0	1.5	13.2	13.3	7.4	7.8
5.5	5.9	3.2	4.0	13.3	13.7	2.1	2.7
3.3	4.1	3.2	3.9	14.0	14.9	8.0	7.8
4.0	4.5	2.5	3.4	12.0	13.3	2.0	2.7
4.8	5.3	2.1	3.0	13.5	14.0	6.2	6.6
6.0	6.4	1.4	2.0	14.1	15.6	7.5	7.5
7.1	7.1	1.0	1.4	15.6	15.8	1.5	1.8
6.5	7.2	0.7	1.0	15.0	16.0	7.4	8.1
		0.7	0.8	15.4	16.5	2.0	2.0
		0.7	0.8	6.4	7.4	7.7	8.6
				6.1	7.1	1.8	2.1
				6.2	6.9	5.5	6.6
						1.5	1.7

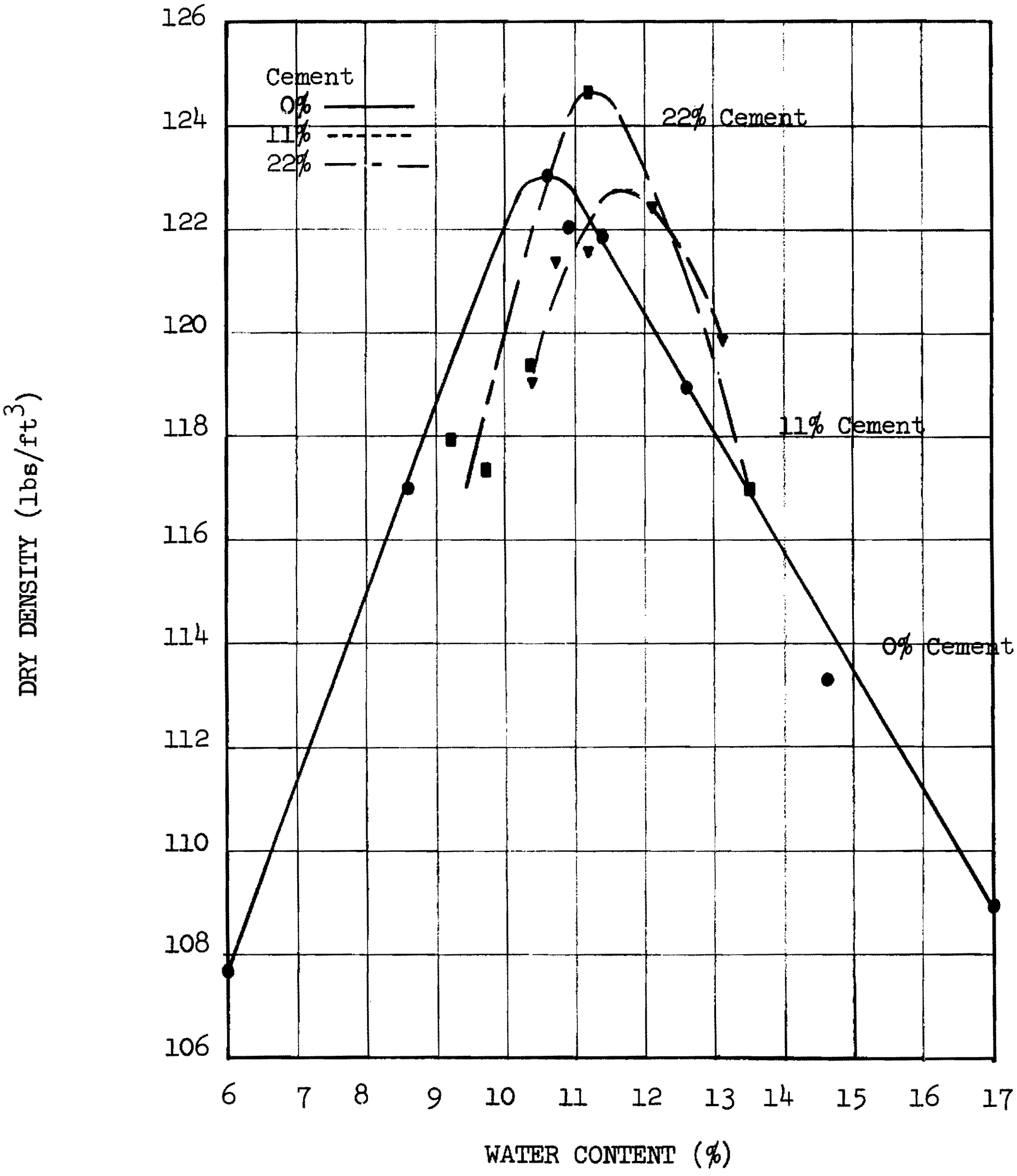

Figure 33. Moisture-Density Relationship Soil A.

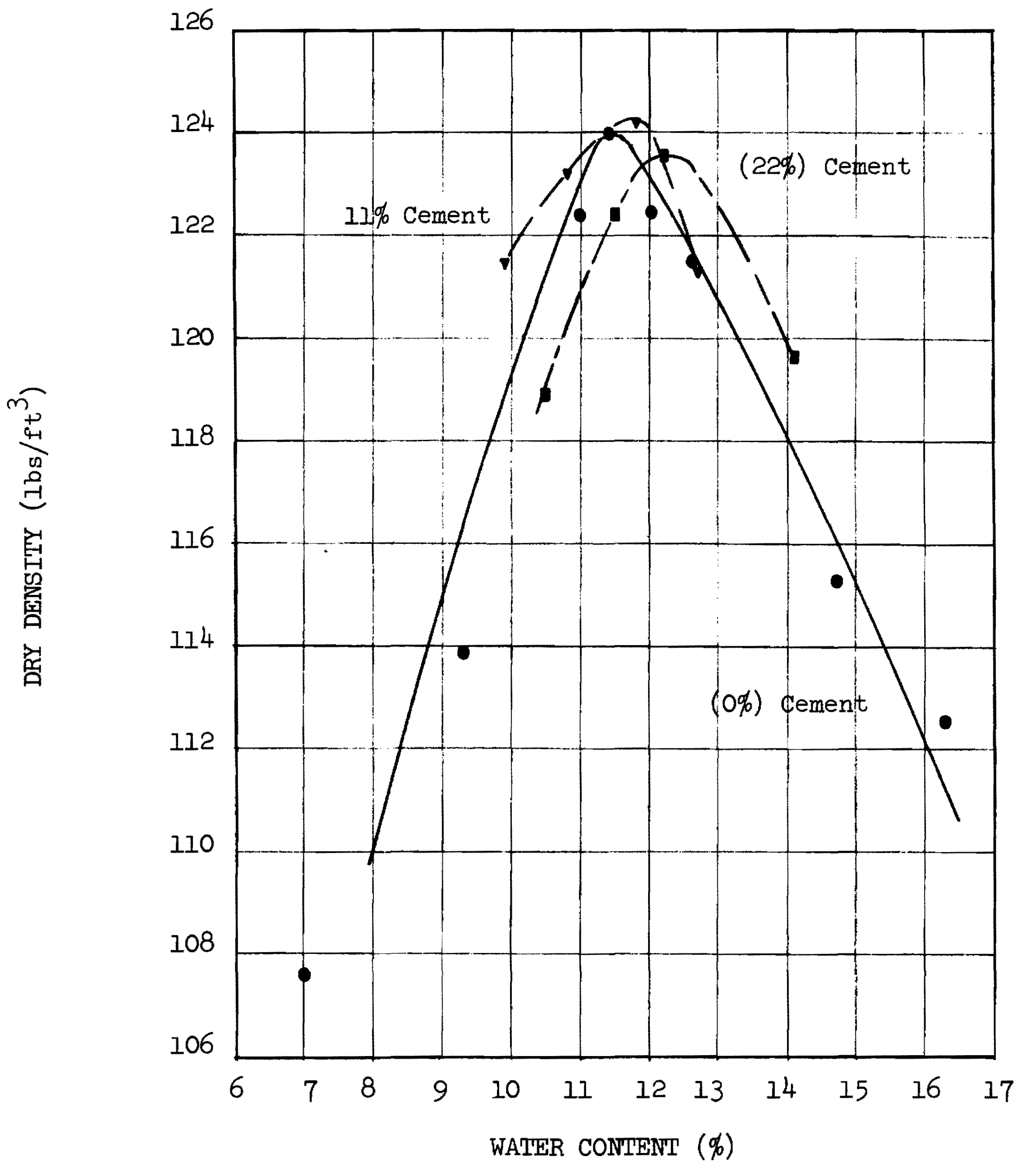

Figure 34. Moisture-Density Relationship Soil B.

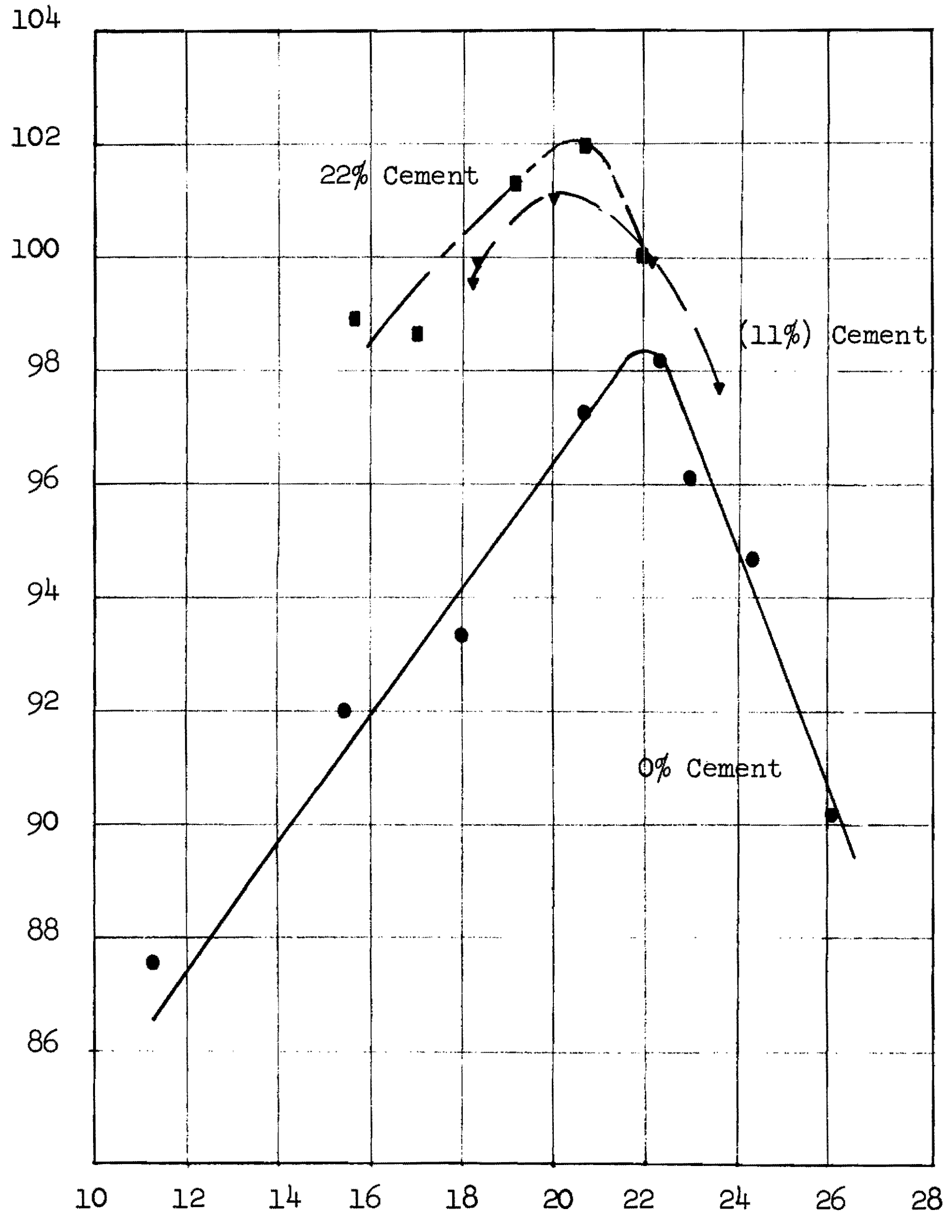

Figure 35. Moisture-Density Relationship Soil C.

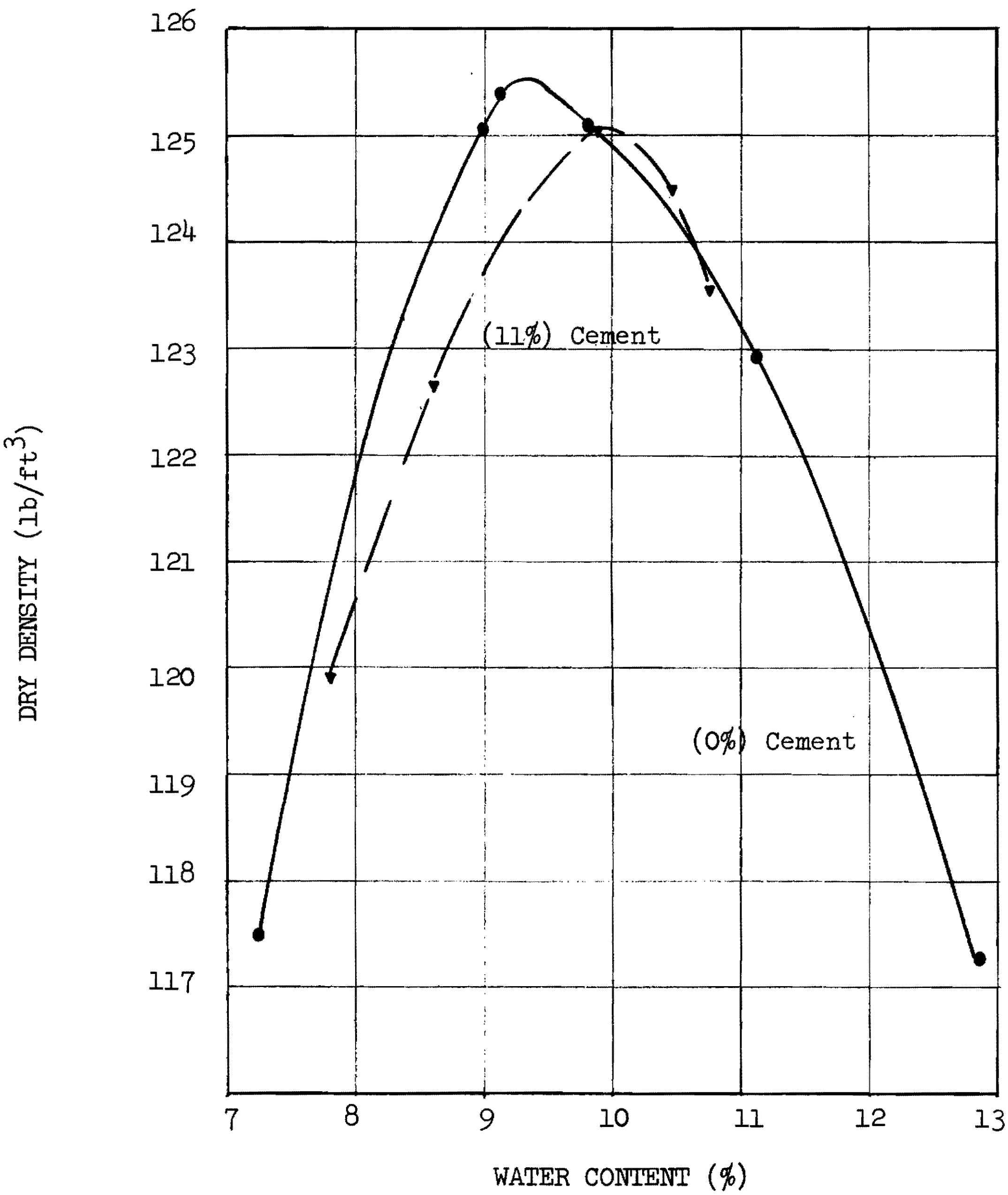

Figure 36. Moisture-Density Relationship Soil D.

Curing of Specimen

The objective of this study was to develop a curing process that would be as near ideal as possible, to permit the specimen to be accessible for numerous observations, and show any cracks that may develop.

Several methods were tried, such as coating the entire specimen with a thin coating of RC-3; placing specimen in plastic freezer bags; and wrapping the specimen with a tight fitting saran wrap. The RC-3 coating was too time consuming and would not produce the proper water retention in the specimen. The plastic freezer bags were too loose fitting, resulting in poor observation of the specimen. The saran wrap, which was selected for this experiment, kept the moisture content constant in the specimen, involved a minimum of time in placing and due to its clear, glass-like and tight fitting properties permitted excellent observation of the specimen. This wrapping process is shown in Figure 37.

Testing Relationships

With the information obtained from the pilot studies, a testing procedure was followed which would determine what the effect of varying certain factors would have upon the cracking of the cement treated soil bases, namely would varying these factors increase, decrease or not affect cracking.

The factors used were clay content present in the soil, cement content, moisture content, curing, temperature differential existing in bases and aggregate content in macadam bases. Relationship of these factors to cracking are as follows:

Clay Content Present in the Soil

The higher the clay content of a soil the more that soil would behave like a clay, that is, it would have a large volume change and a great affinity for water. The ability of the clay to keep the water within itself

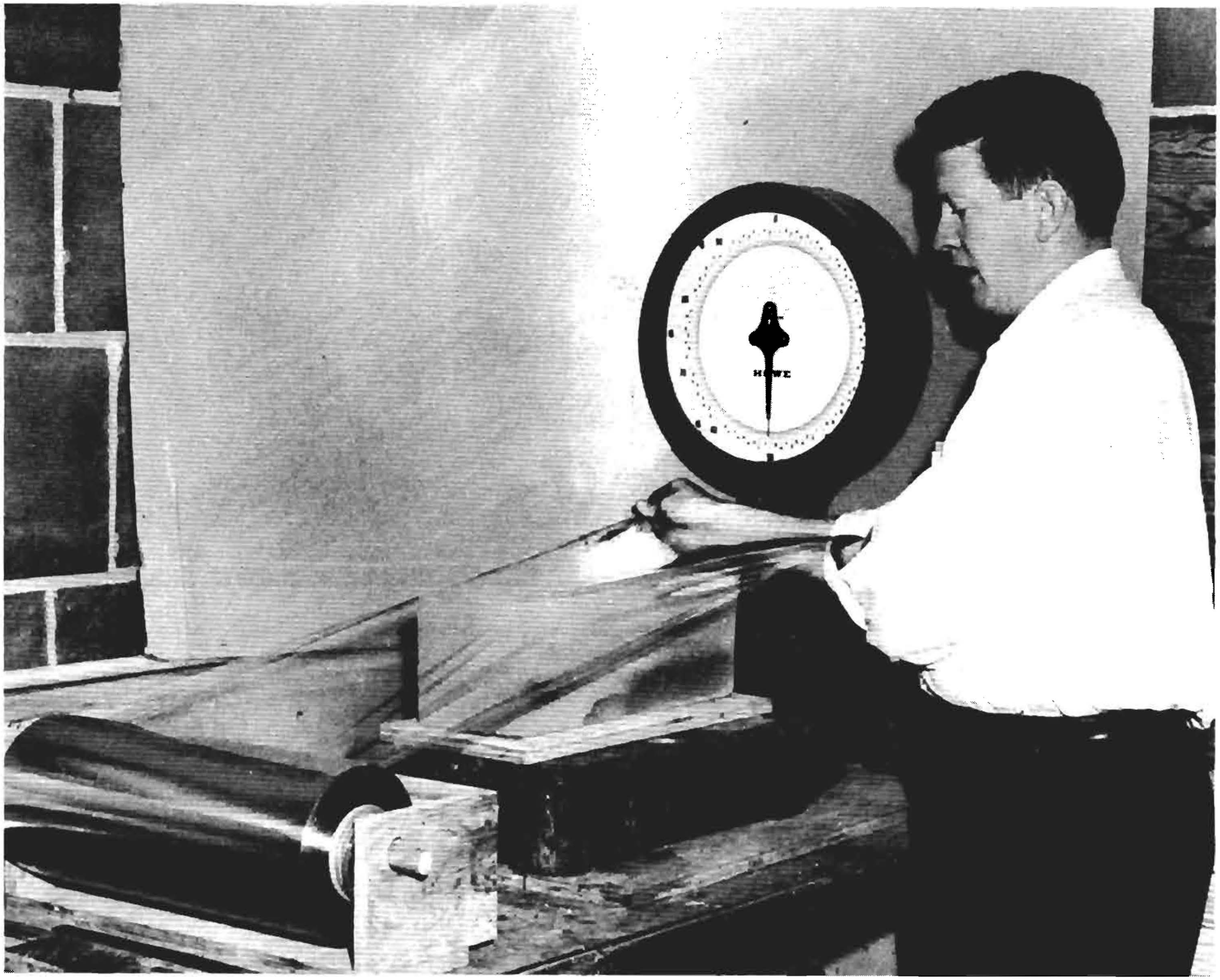

Figure 37. Wrapping Process for Curing.

would thereby not permit sufficient cement hydration and upon the loss of water the soil would not be strong enough to resist the volume change. This could cause cracking.

Cement Content

Due to the autogenous shrinkage of Type I Portland Cement, the higher the cement content the larger would be the volume loss which would result in cracking. Also, during and after the hydration process of the cement, the cement has an affinity for water similar to that of clay.

Moisture Content

The ideal moisture content of different soils and cement combinations for reducing cracking is not always the optimum found by the moisture-density relationship.

Curing

The loss of moisture by evaporation will cause a decrease in volume which results in cracking. Keeping the moisture in the soil, by proper curing would reduce cracking.

Temperature Differential

The temperature differential that exists in freshly placed bases in the field, which is around 70°F, could cause cracking. This could be attributed to numerous factors:

1. Accelerated hydration on the top due to the high temperatures.

2. Expansion and contraction properties of the soil resulting in a warping action.

3. The high temperatures on the top forcing the moisture to a cooler region, i.e., the bottom.

4. The pore pressures being greater at the top due to the high temperatures expanding the moisture.

Aggregate Content

If the soil used to fill the voids of the aggregate fills more than 95% of these voids, the soil-bound macadam would tend to possess characteristics similar to that of the soil used.

Testing Procedures

Phase I

This phase of the testing was to find the relationship of clay, moisture, and cement contents and curing. The following test procedure was used:

1. For each of the four soils, two specimens, one 3% below optimum moisture content and one 3% above optimum moisture content, were compacted for each of 10 different cement contents. Cement percentages used were 0, 1, 3, 5, 8, 11, 14, 18, and 22.

2. After compaction, the specimen was trimmed, removed from the mold and wrapped with saran wrap for curing.

3. The specimens were then placed in curing racks in the laboratory and observed at specified periods to determine the presence of and/or the development of cracking. Comparison of the amount of cracking between different specimens was made by photographs.

4. The specimens were retained inside the laboratory for a period of not less than 5 weeks. After this curing period, the saran wrap was removed and the specimens were placed outside and subjected to weathering.

Phase II

Phase II of the testing was to find the relationship of temperatures differential to cracking. The following test procedure was used:

1. For each of the four soils, duplicate specimens were compacted at optimum moisture content and 3% above optimum moisture content, for each of the four cement contents, 0, 3, 8, and 22%.

2. The specimen were trimmed, removed from the mold. One of each duplicate specimen was wrapped with saran wrap while the other was left uncured.

3. The specimen were then placed in the temperature differential apparatus, and subjected to a temperature differential of 70°F from the top to the bottom of the specimen.

4. After a period of 24 hours the specimen were removed from the temperature differential apparatus and photographs were made of the specimens.

Phase III

The influence of stone screenings on cracking was tested in the following manner:

1. The proportion of stone screenings and soil to be used was determined by "the Sower's Method"[31] and the "60-40% Method".

The Sower's Method first divides the soil into aggregate and binder. Then each are compacted separately and from the combined aggregate the volume of voids is found. Then from these voids the amount of binder needed to fill them is determined. The addition of 90% of this needed binder is made to the aggregate.

The "60-40% Method" is a mixture of 60% aggregate and 40% binder, assuming the specific gravity of the aggregate is about 2.65.

2. The remainder of the test procedure followed that of Phase II.

Preparation

1. The amount of soil needed was calculated from moisture-density data, removed from storage barrels, and placed in the mixing bowl. The soil was then mixed until homogeneous.

2. The hygroscopic moisture content of the soil was estimated by the use of a "speedy" moisture tester manufactured by the Alpha Lux Company, Inc.

At this same time a moisture sample of the soil was placed in a 10 oz. can, dried in an oven at 230°F for 24 hours and the actual water content determined. If the actual hygroscopic moisture content was more than $\pm$ 1.0 different from the "speedy" reading, the specimen was discarded and a new one made.

3. The derived amount of cement was then weighted to the nearest 0.01 pound, and mixed with the soil until homogeneous. The amount of cement used was calculated as a per cent of the weight of dry soil. For example, if the weight of the dry soil was 50 lbs. and 5% cement content was desired, 2.50 lbs. of cement would be added.

4. The amount of water to be added was weighed to the nearest 0.01 pound and slowly added to the soil-cement mixture. The amount of water used was calculated as a per cent of the total dry weight of the soil and cement. For example, if the weight of the dry soil was 50 lbs. the weight of cement was 2.50 lbs. and the desired water content was 10%, then 5.25 lbs. of water would be needed.

5. The contents of the bowl was then mixed by the Read Standard Grant mixer for a period of 45 seconds. The blade and sides of the bowl were scraped and the contents mixed for another 45 seconds.

Compaction

1. The amount of the soil-cement mixture needed for a two inch layer was calculated and placed in the mold. The soil was compacted by allowing an eleven pound rectangular hammer to be dropped 123 times from a height of 12 inches above the surface of the soil (See Fig. 26 for details of the compaction equipment.) This procedure was repeated for each of the three layers, but allowing for enough excess on the top layer to be scraped off level with the top of the mold. During compaction the mold was moved hori-

zontally to insure that each layer would be 2 inches thick.

3. The compacted sample was removed from the mold, placed on a 3/4" x20"x9" plywood board and weighted to the nearest 0.1 pound. To simulate curing, the sample was completely sealed by wrapping it with transparent saran wrap. The samples were placed in storage bins inside the laboratory for periodical observation as shown in Figure 38.

Test Results

The results obtained from Phase I were both numerous and conclusive. During a five week curing period with observations being made periodically, no evidence of cracking was detected in the specimens. It was concluded, therefore, that the internal action of Type I Portland Cement does not by itself cause cracking. Pictures made of these specimens immediately after the curing was removed are shown in Figures 39 through 41.

From the periodical observations, it was found that with increasing cement contents more moisture appeared to be retained within the specimen by proportionality. The low cement contents (0-3%) specimens showed excessive amounts of moisture collected on the inside of the curing material. These findings were similar for all four soils used in this test.

After removal of the curing jackets, all of the four different soil specimens with low cement contents and moisture content 3% above optimum, cracked within 24 hours.

Observations made of the specimens while being exposed to weathering showed the moisture content, whether it was 3% above or 3% below optimum, as being very critical to the durability of the specimen. The clay soils with high cement contents were unaffected by the heavy rains if they were compacted at 3% above optimum moisture content but were very badly pitted if they were compacted 3% below optimum moisture content. For the low cement

Figure 38. Storage Bins.

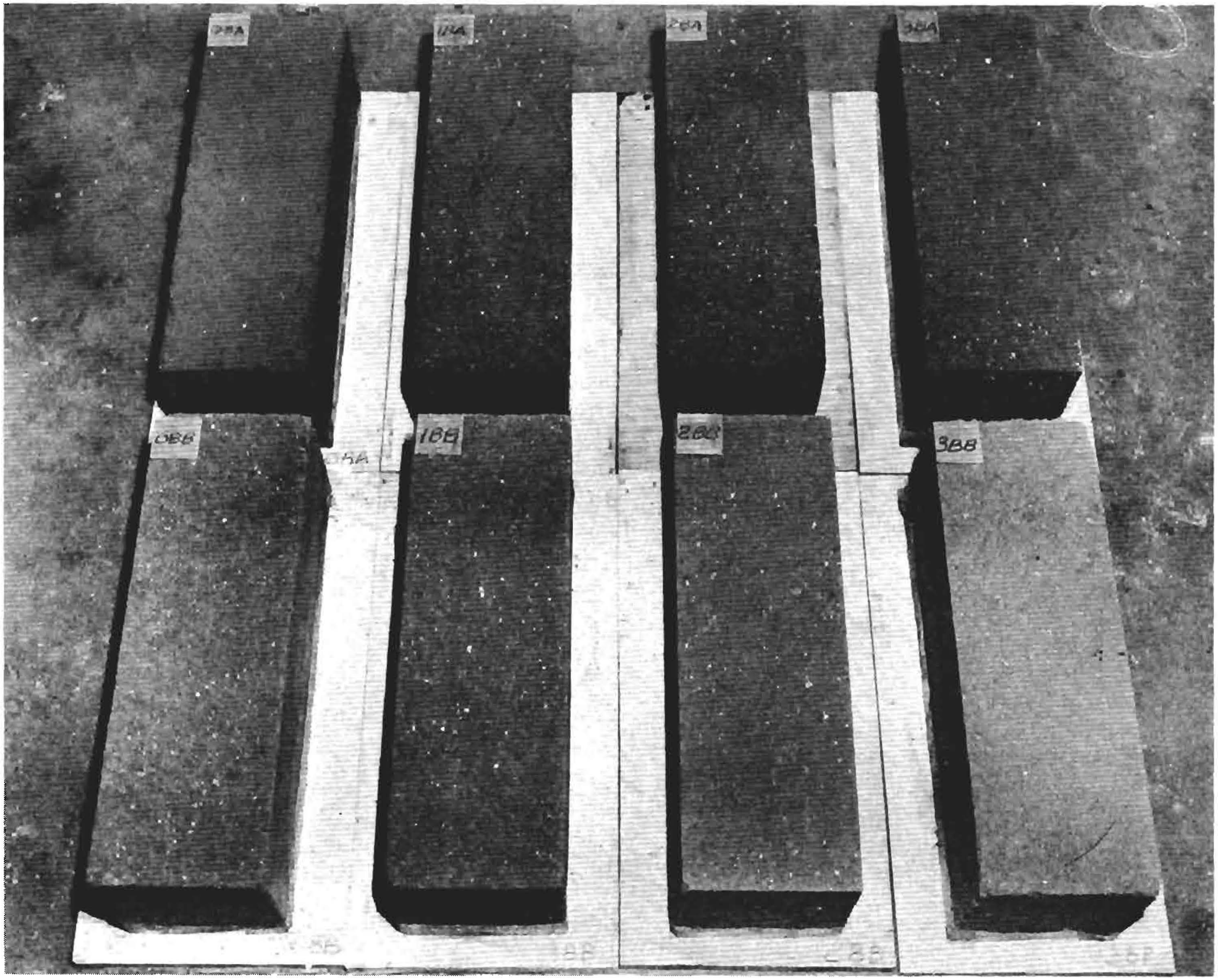

Figure 39. Phase No. I Specimens for Soil B with 0, 1, 2, and 3 Per Cent Cement Content.

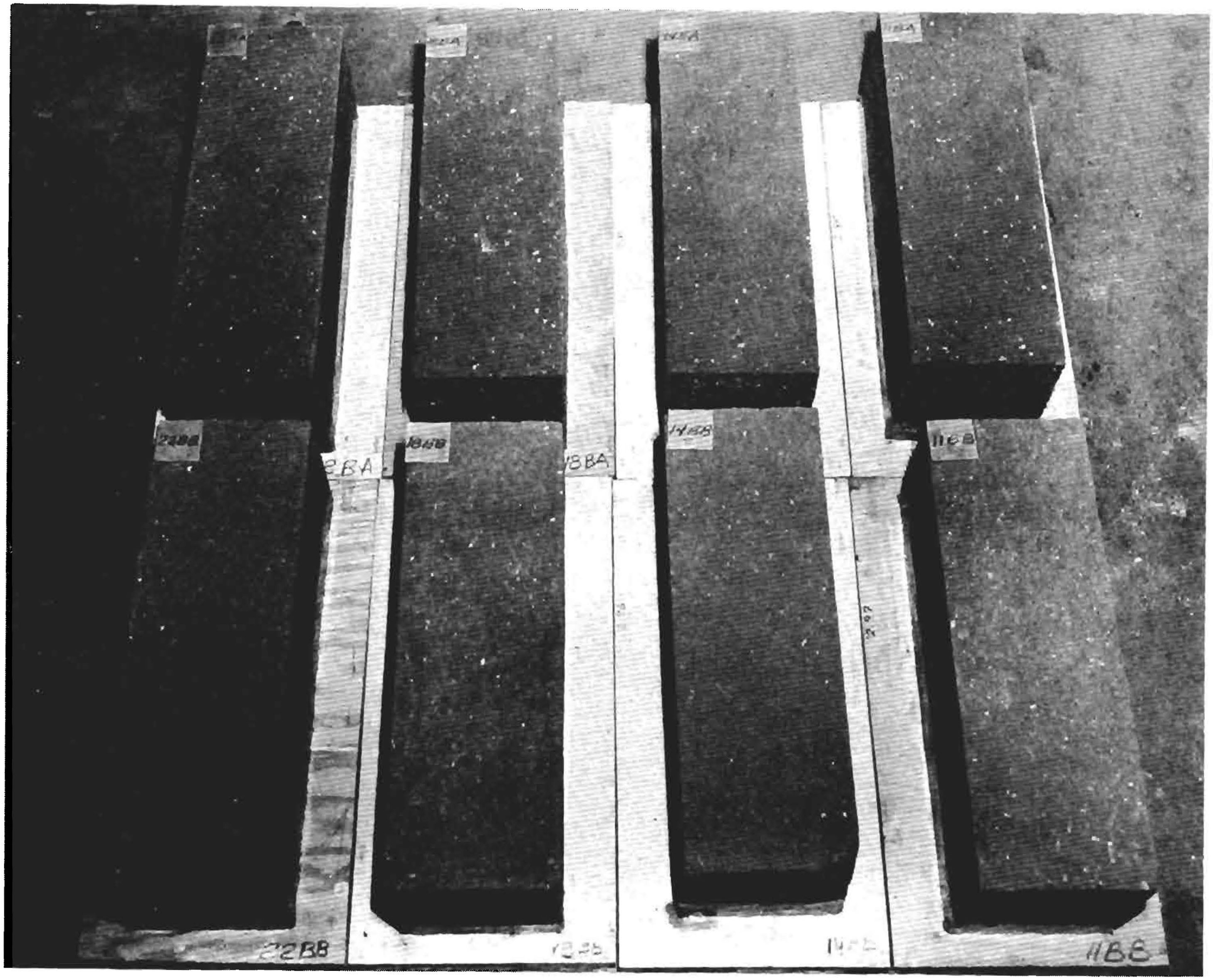

Figure 40. Phase No. I Specimens for Soil B with 11, 14, 18, and 22 Per Cent Cement Content.

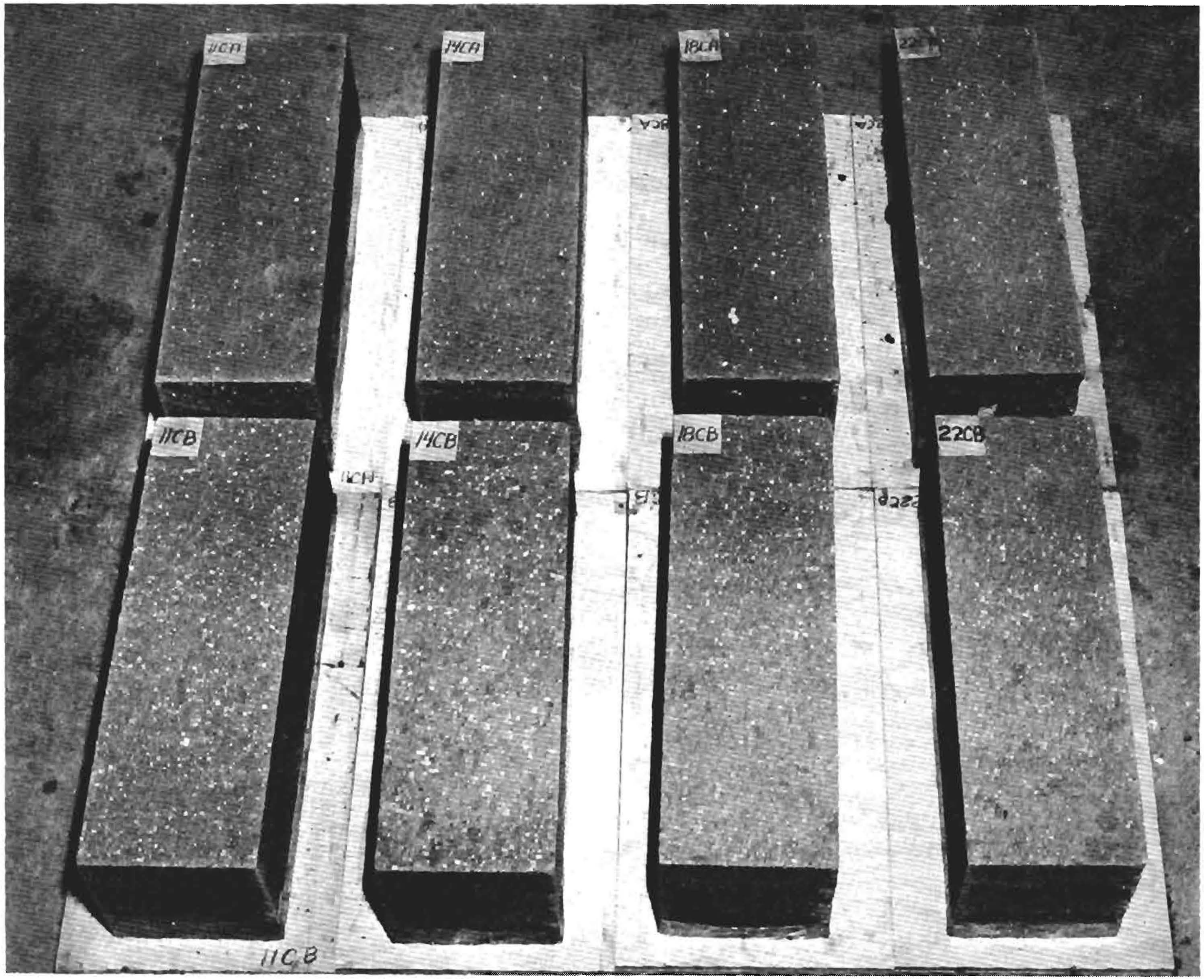

Figure 41. Phase No. I Specimens for Soil C with 11, 14, 18, amd 22 Per Cent Cement Content.

contents the reverse was true except not nearly as noticeable. The more friable soils showed this same relationship but in a much smaller degree. Pictures of these samples are shown in Figures 42 through 44.

This same relationship was found from research on permissible moisture content variation from optimum during the year 1938. The most important point shown by this early research is the following conclusion taken from the report.[33]

> "The optimum moisture content (Point at which maximum density is obtained) as shown by the standard moisture density test is reasonably in agreement with the optimum moisture contents at which maximum durability and maximum strength are obtained. For sandy soil-cement mixtures, moisture contents, at the optimum (for maximum density) are ideal for best all-around durability and strength; but for silt soils and clay soils the moisture content for best all-around durability and strength is slightly wetter than the optimum quantity."

Graphs plotted from values given in Tables XXVII through XXXII are shown in Figures 45 through 48. These graphs show the dry density remaining almost constant for each soil with an increasing cement content when compacted 3% above optimum moisture content. When the soils were compacted 3% below optimum moisture content, each soil, except soil C, showed a decrease in dry density with increasing cement contents and then a slight increasing in dry density with further increasing in cement content. Soil C, due to its light weight characteristics, was very sensitive to changing cement contents thereby resulting in widely varied dry densities. These graphs illustrate further the detrimental effect of varying moisture contents with different cement contents.

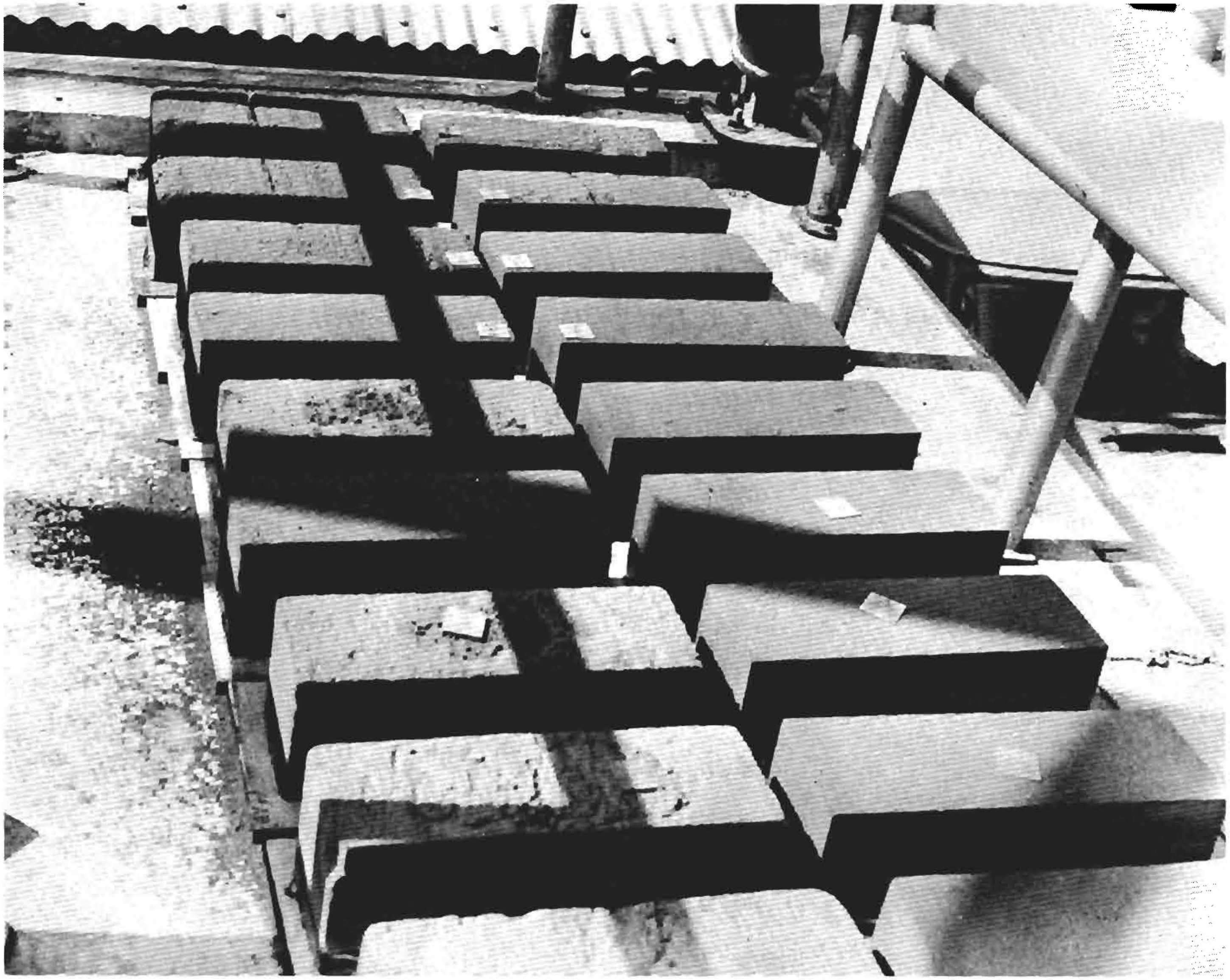

Figure 42. Phase No. I Specimens with Outside Exposure for Soil A.

Figure 43. Phase No. I Specimens with Outside Exposure for Soils B and C.

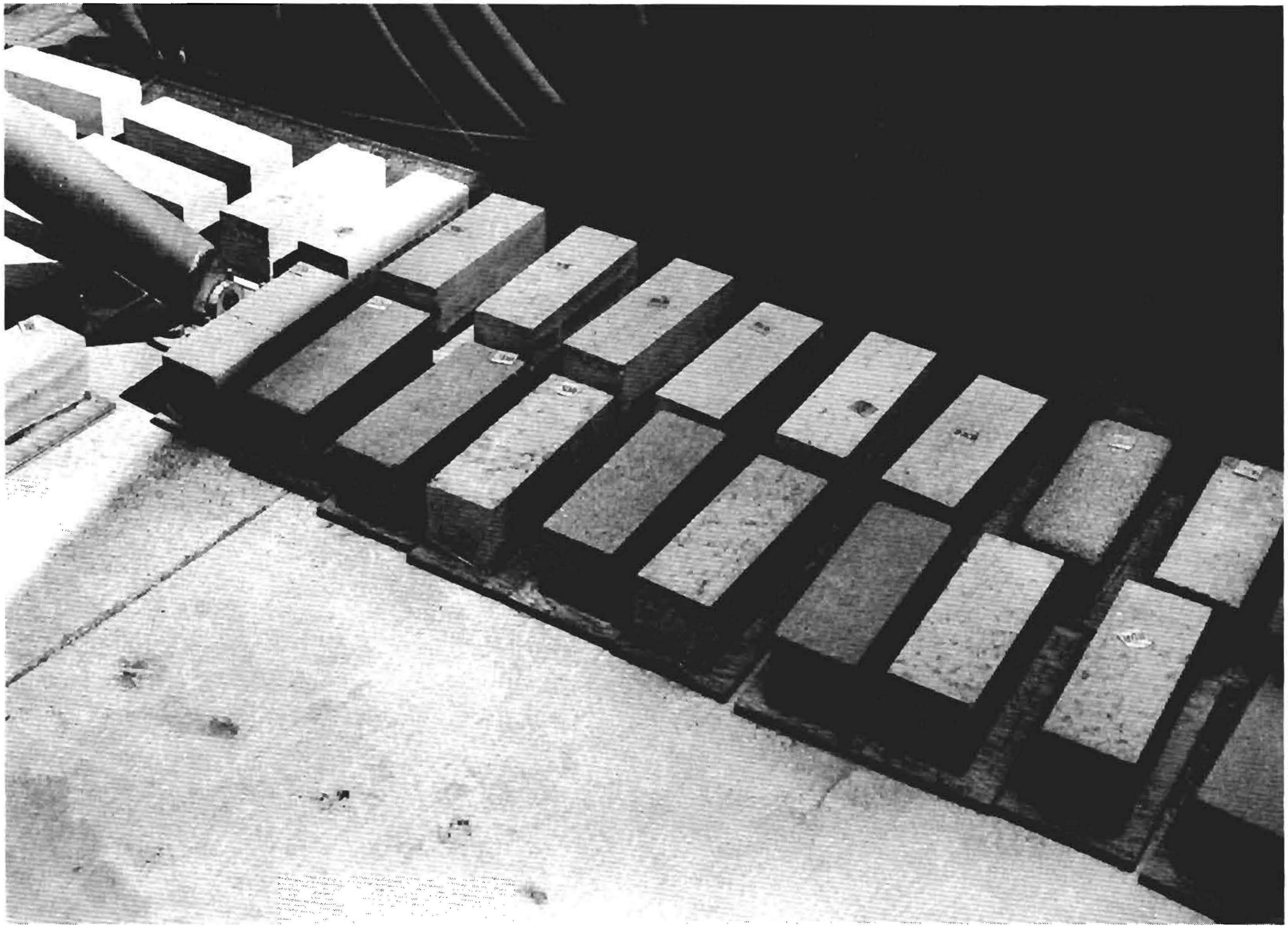

Figure 44. Phase No. I Specimens with Outside Exposure for Soils C and D.

Table XXVII. Phase No. 1 Specimen Data (Soil A)

Specimen No.	% Cement	Water Content Desired	Water Content Actual	Density Wet	Density Dry
OAB	0	7.6	7.6	128.7	119.6
OAA	0	13.6	13.1	136.3	120.5
1AB	1	7.6	7.6	122.7	114.0
1AA	1	13.6	13.6	134.9	118.8
2AB	2	7.6	8.0	115.3	106.8
2AA	2	13.6	13.5	134.4	118.4
5AB	5	7.6	7.6	114.9	106.8
5AA	5	13.6	13.9	134.2	117.8
8AB	8	7.6	7.6	106.5	99.6
8AA	8	13.6	13.5	136.5	120.3
11AB	11	7.6	7.7	112.6	104.5
11AA	11	13.6	13.1	127.9	113.1
14AB	14	7.6	7.3	106.4	99.2
14AA	14	13.6	13.7	133.9	117.8
18AB	18	7.6	7.1	106.0	99.0
18AA	18	13.6	13.1	137.0	121.1
22AB	22	7.6	7.4	104.8	97.6
22AA	22	13.6	12.7	134.1	119.0

Table XXVIII. Phase No. 1 Specimen Data (Soil B)

Specimen No.	% Cement	Water Content		Density	
		Desired	Actual	Wet	Dry
0BB	0	8.4	8.1	126.0	116.6
0BA	0	14.4	14.1	137.0	120.1
1BB	1	8.4	8.3	125.0	115.4
1BA	1	14.4	14.1	137.7	120.7
2BB	2	8.4	8.0	125.1	115.8
2BA	2	14.4	14.2	137.1	120.1
3BB	3	8.4	7.7	123.0	114.2
3BA	3	14.4	13.5	137.6	121.2
5BB	5	8.4	7.6	118.2	109.9
5BA	5	14.4	13.5	137.6	121.2
8BB	8	8.4	7.7	122.7	113.5
8BA	8	14.4	13.8	138.9	122.1
11BB	11	8.4	8.6	128.1	118.0
11BA	11	14.4	14.4	138.3	120.9
14BB	14	8.4	7.9	122.8	113.8
14BA	14	14.4	14.5	138.4	120.9
18BB	18	8.4	8.8	126.7	116.5
18BA	18	14.4	14.8	138.0	120.2
22BB	22	8.4	8.9	127.4	117.0
22BA	22	14.4	14.8	136.8	119.2

Table XXIX. Phase No. 1 Specimen Data (Soil C)

Specimen No.	% Cement	Water Content Desired	Water Content Actual	Density Wet	Density Dry
0CB	0	19	14.6	106.8	91.7
0CA	0	25	24.0	118.3	95.9
1CB	1	19	20.0	115.7	96.0
1CA	1	25	26.0	120.0	94.8
2CB	2	19	20.0	116.3	96.9
2CA	2	25	25.2	120.2	96.0
3CB	3	19	20.0	117.1	97.6
3CA	3	25	26.0	121.0	96.0
5CB	5	19	19.7	122.0	101.9
5CA	5	25	26.0	120.4	95.6
8CB	8	19	20.0	119.8	99.8
8CA	8	25	26.0	121.2	96.2
11CB	11	19	19.5	113.6	95.1
11CA	11	25	26.0	121.0	96.0
14CB	14	19	19.8	115.8	96.7
14CA	14	25	26.0	120.8	95.9
18CB	18	19	19.1	117.2	98.4
18CA	18	25	26.0	120.5	95.2
22CB	22	19	19.9	113.6	94.7
22CA	22	25	25.4	122.5	97.7

Table XXX. Phase No. 1 Specimen Data (Soil D)

Specimen No.	% Cement	Water Content Desired	Water Content Actual	Density Wet	Density Dry
ODB	0	6.5	7.1	130.4	121.8
ODA	0	12.5	12.7	136.5	121.1
IDB	1	6.5	7.1	128.1	119.6
IDA	1	12.5	12.7	136.5	121.1
2DB	2	6.5	7.0	126.3	118.0
2DA	2	12.5	12.4	137.0	121.9
3DB	3	6.5	6.9	125.6	117.5
3DA	3	12.5	12.9	136.5	120.9
5DB	5	6.5	7.1	125.0	116.7
5DA	5	12.5	12.3	136.9	121.9
8DB	8	6.5	7.2	122.3	114.1
8DA	8	12.5	12.9	138.1	122.3
11DB	11	6.5	6.8	121.5	113.8
11DA	11	12.5	12.5	138.2	122.8
14DB	14	6.5	6.5	123.1	115.6
14DA	14	12.5	13.2	137.6	121.6
18DB	18	6.5	6.8	121.5	113.8
18DA	18	12.5	13.4	136.5	120.4
22DB	22	6.5	6.7	124.3	116.5
22DA	22	12.5	13.5	138.4	121.8

Table XXXI. Phase No. 2 Specimen Data (Soils A & B)

Specimen No.	% Cement	Water Content		Density	
		Desired	Actual	Wet	Dry
0A	0	10.6	11.7	131.4	117.6
0A	0	10.6	11.4	130.8	117.4
3A	3	10.6	11.1	131.3	118.2
3A	3	10.6	10.1	124.8	113.4
8A	8	10.6	10.3	126.6	114.8
8A	8	10.6	9.8	118.3	107.7
8A	8	13.6	13.1	134.1	118.6
8A	8	13.6	12.7	132.1	117.2
0B	0	11.4	11.4	136.5	122.5
0B	0	11.4	11.4	138.7	124.5
0B	0	14.4	14.4	136.0	118.9
0B	0	14.4	14.4	134.5	117.6
3B	3	11.4	11.5	135.7	121.9
3B	3	11.4	11.7	136.7	122.4
8B	8	11.4	11.5	138.1	123.9
8B	8	11.4	11.5	131.7	118.1
8B	8	14.4	14.4	137.5	120.2
8B	8	14.4	14.8	138.5	120.6

Table XXXII. Phase No. 2 Specimen Data (Soils A, B, C & D)

Specimen No.	% Cement	Water Content Desired	Water Content Actual	Density Wet	Density Dry
OC	0	22	22.9	119.6	97.3
OC	0	22	22.6	121.5	99.1
OC	0	25	25.8	116.7	92.8
3C	3	25	25.4	120.5	96.1
8C	8	22	22.3	121.7	99.5
8C	8	25	25.2	121.9	97.4
3C	3	25	25.5	120.4	95.9
OC	0	25	26.0	119.0	94.4
OD	0	9.5	10.1	138.9	126.2
OD	0	12.5	13.2	135.6	119.8
3D	3	9.5	10.3	136.8	124.0
3D	3	12.5	13.5	137.0	120.7
8D	8	12.5	12.9	137.4	121.7
8D	8	9.5	9.9	134.3	122.2
22C	22	25	25.9	122.2	97.1
22A	22	13.6	13.9	136.1	119.5
22B	22	14.4	14.7	134.9	117.6
22D	22	12.5	12.8	139.0	123.2

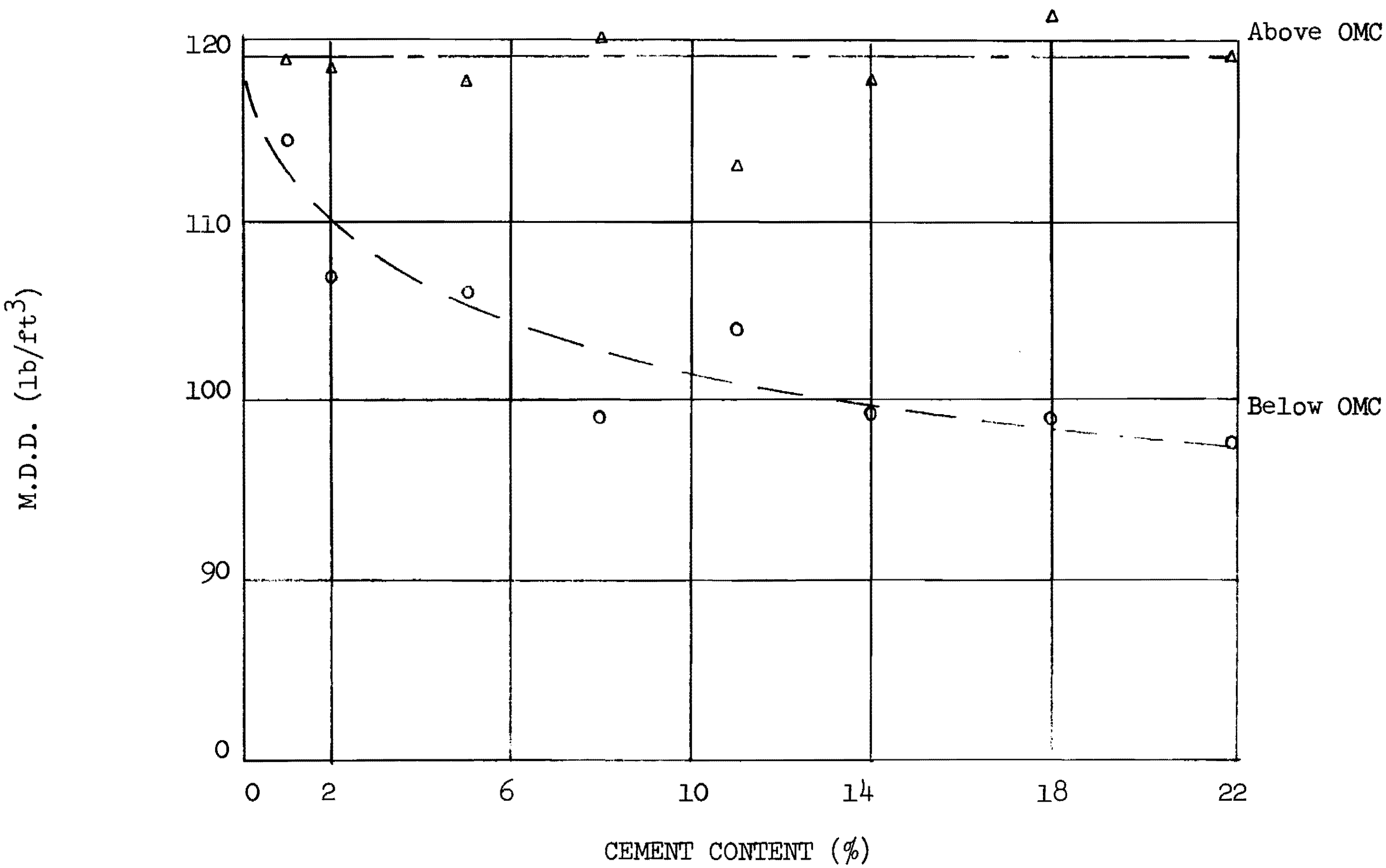

Figure 45. Variance of Maximum Dry Densities for Soil A.

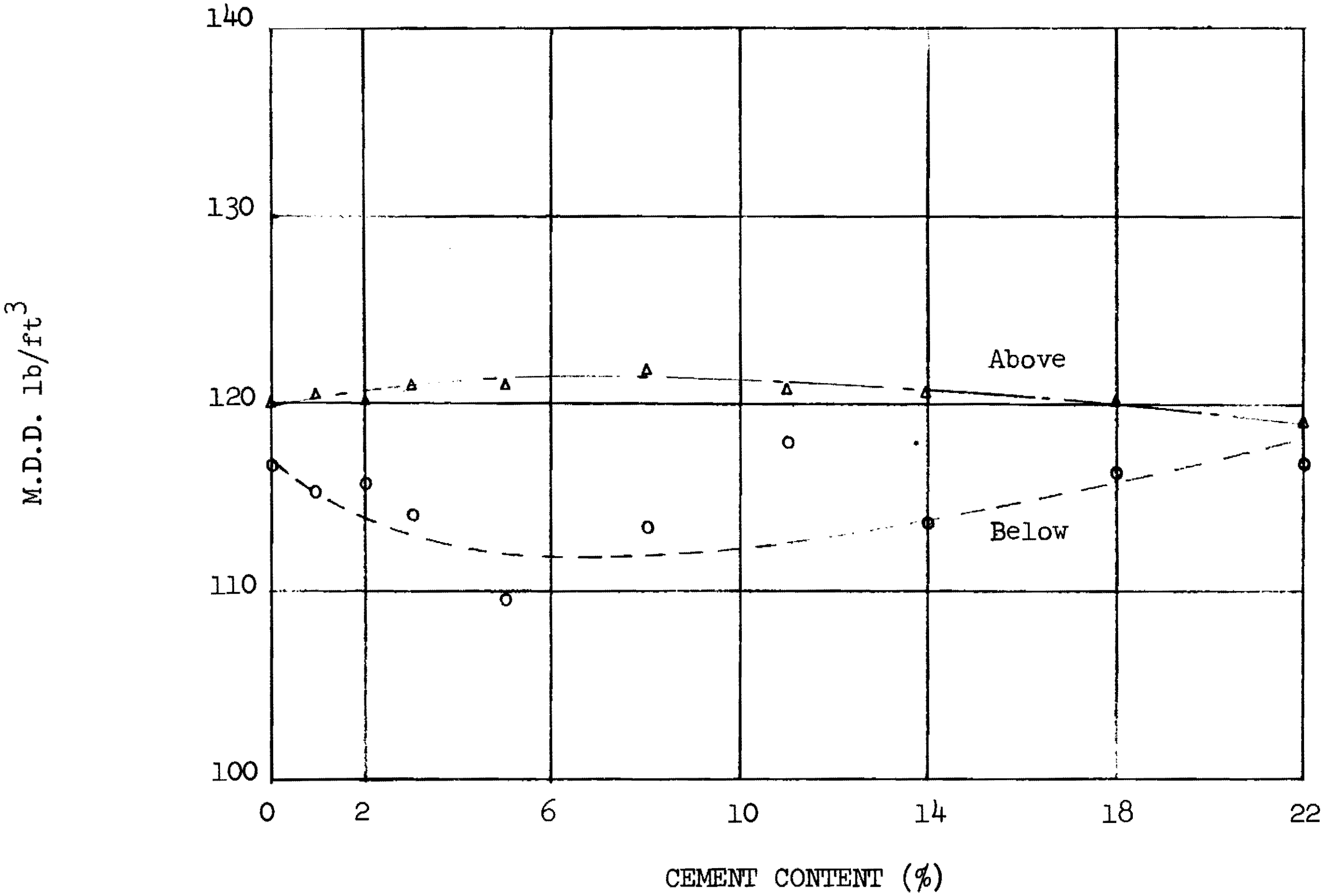

Figure 46. Variance of Maximum Dry Density for Soil B.

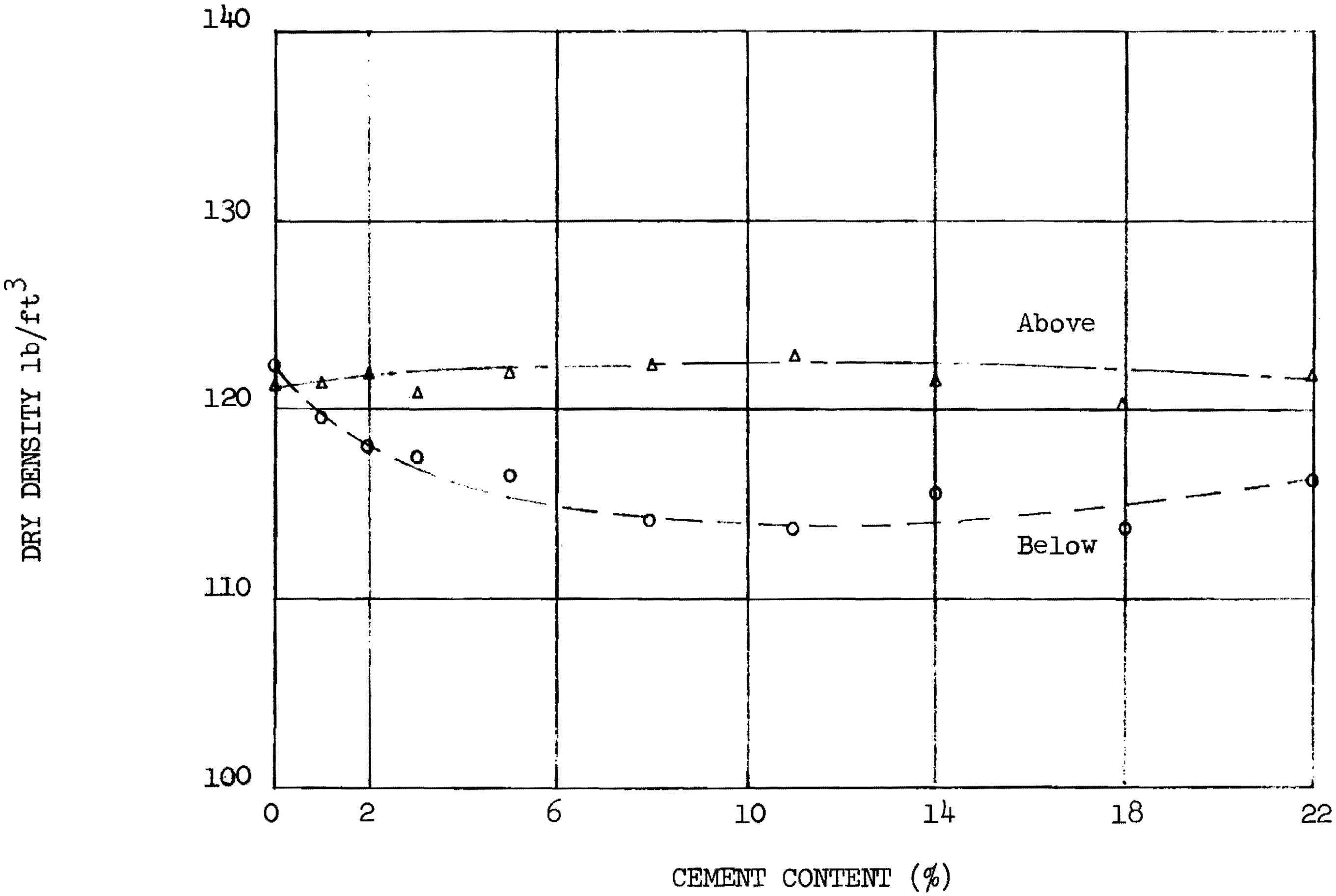

Figure 47. Variance of Maximum Dry Densities for Soil D.

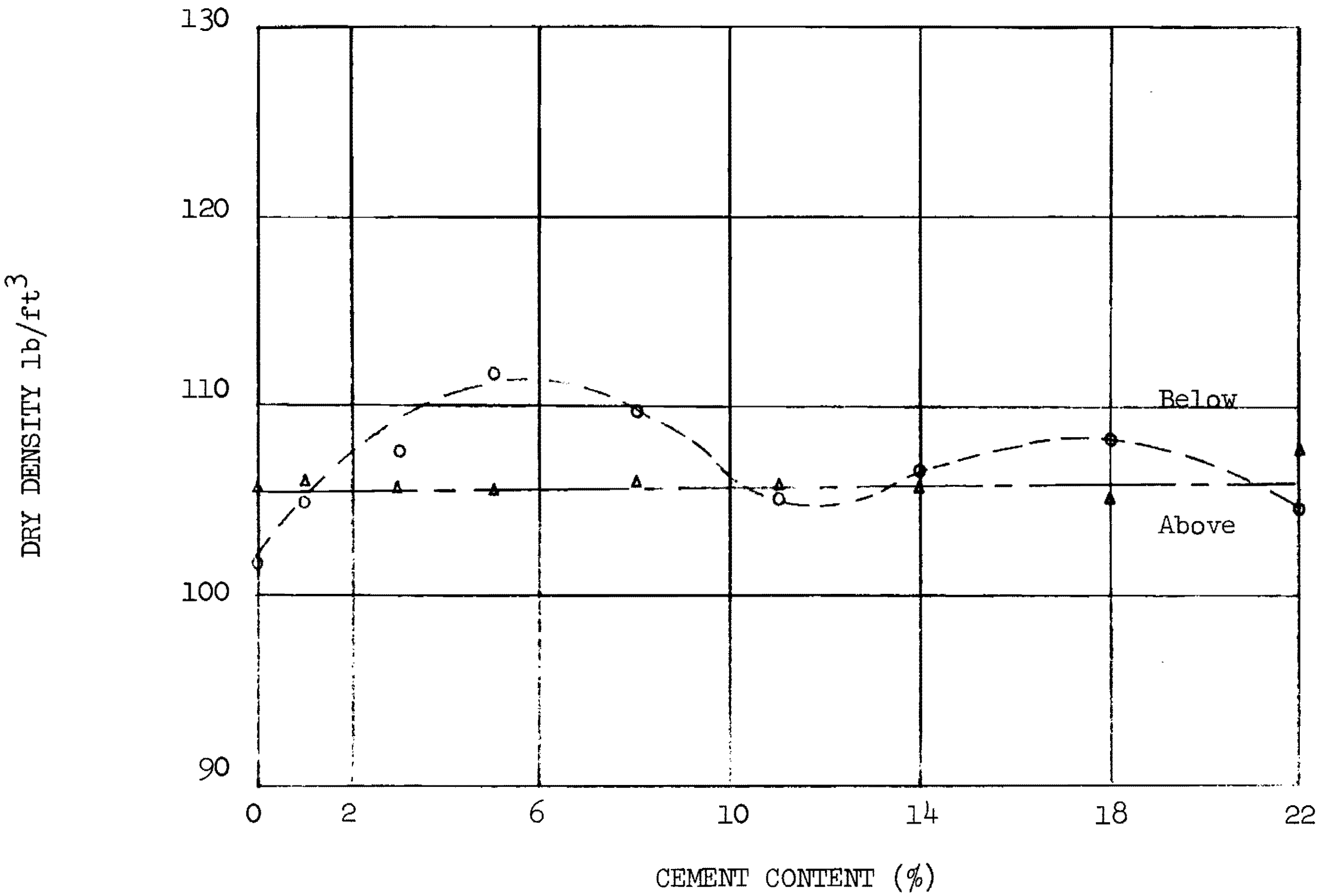

Figure 48. Variance of Maximum Dry Densities for Soil C.

Phase II produced many rewarding results. One of the most astonishing of these was that cracking occurred in specimens which were cured for a 100% moisture retention. There was less cracking observed in the specimens compacted at optimum moisture content. It was also observed that soils with the higher clay content developed the most cracking. After removing the cured specimens from the temperature differential apparatus, it was found in all cases that the moisture content had decreased in the top about 4% and increased at the bottom by about the same amount. This was attributed to the physical characteristics of water which causes migration of the moisture to colder regions. In most cases where two identical specimens, except for curing, were subjected to the same temperature, the uncured specimen developed more accelerated and serious cracking.

Soil A developed no cracks for the different cement contents, cured or not cured, when the specimens were compacted at optimum moisture content. A few cracks did appear in the cured specimen compacted with 8% cement and 3% above optimum moisture content. The identical uncured specimens to the above developed no cracks. The cause of the cracks in the cured specimens was attributed to the pore pressure caused by the expanding water created by the high temperature. Some of these results are shown in Figure 49.

Soil B, compacted at optimum moisture content with 0% cement content, and cured, developed no cracks. The twin to this specimen which was not cured showed large cracks across the top. Holding the cement content constant and increasing the moisture content 3% the same results were observed. These results are shown in Figure 50. Using a 3% cement content and the moisture content at optimum, the compacted specimen cured showed no cracks while the uncured specimens showed one very small crack across the top. Increasing the cement content to 8% and keeping the moisture content at opti-

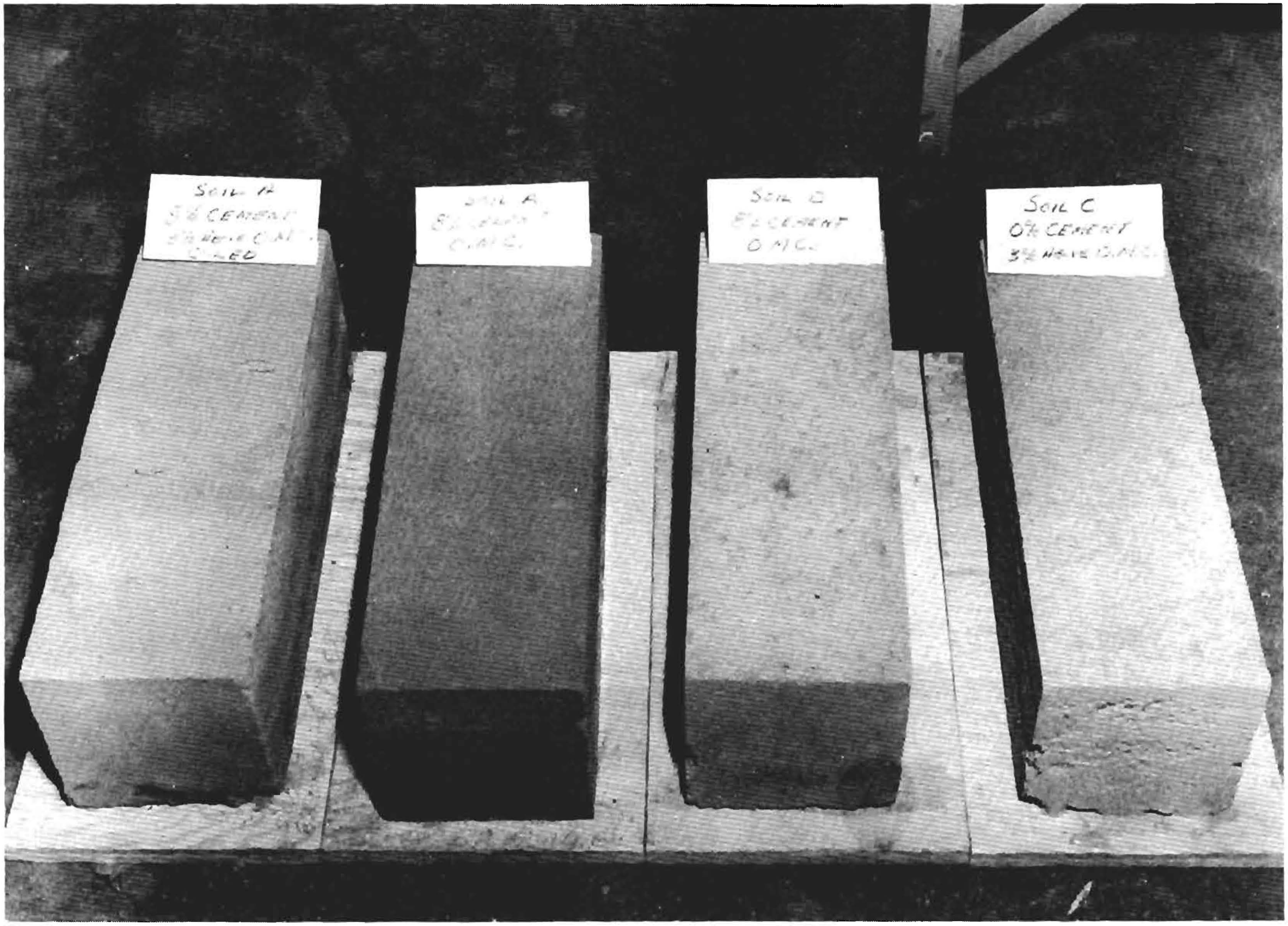

Figure 49. Phase No. II Specimens for Soils A, D, and C.

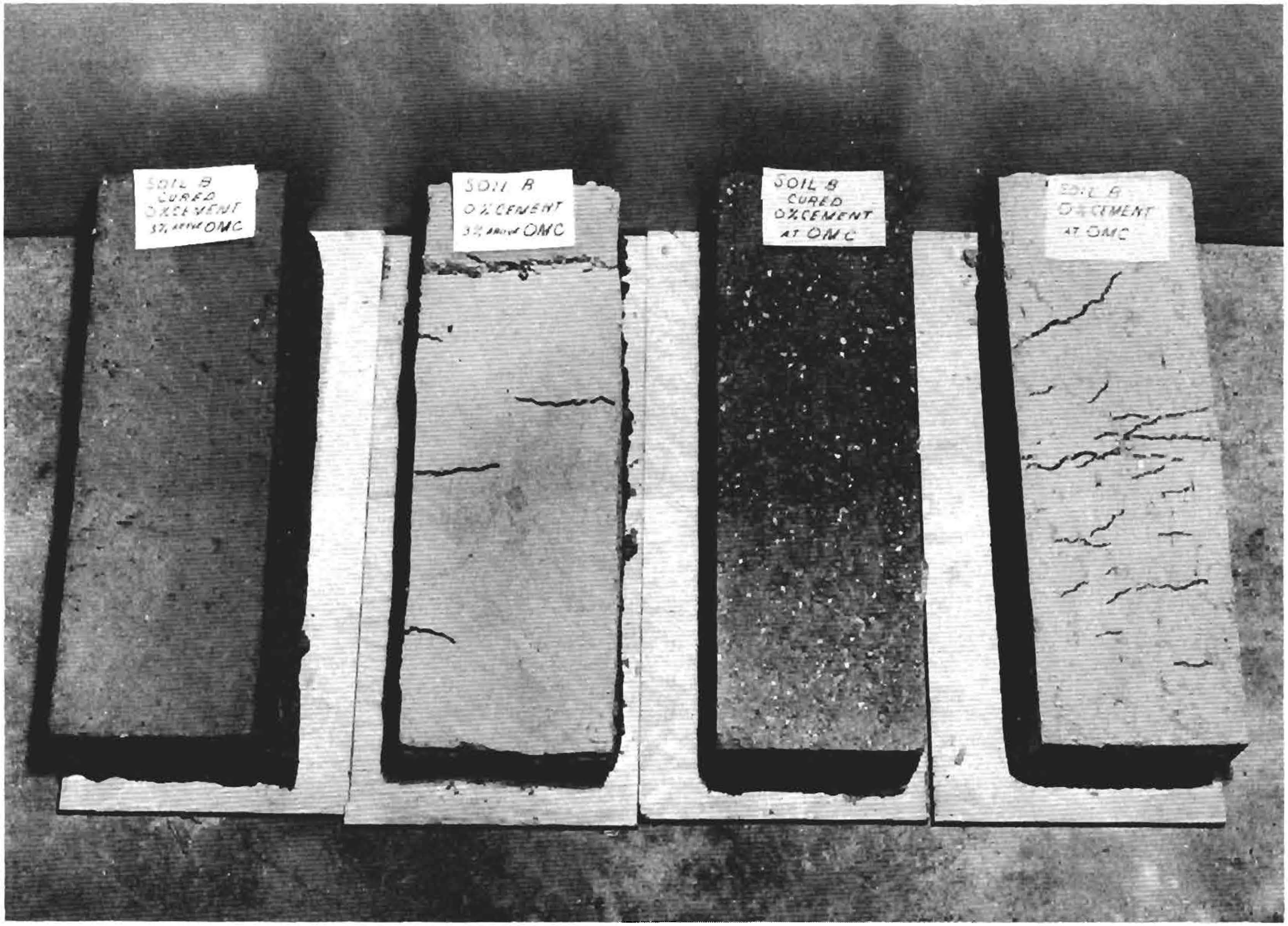

Figure 50. Phase No. II Specimens for Soil B.

mum, the cured specimens developed one crack across the top, perpendicular to the length, about 2 inches deep, while the uncured specimen did not crack. Holding the cement content constant at 8% and increasing the moisture content to 3% above optimum, both of the compacted specimens, cured and non-cured, developed numerous cracks. Also, both specimens developed one crack about two inches deep across the top at mid-point. Pictures of these specimens are shown in Figure 51.

No cracks developed in Soil C for the varied cement contents when compared at optimum moisture content. Cracks did appear though, in every case for the 0, 3 and 8% cement contents when compacted 3% above optimum moisture content. These specimens are shown in Figure 51.

The only cracks that developed while using Soil D were the specimens compacted at 0 and 3% cement content with a moisture content of 3 per cent above optimum. The specimen with 3% cement developed two cracks across the top about two inches deep at the mid-point. These specimens are shown in Figure 52.

Figure 53 shows all the four different soils compacted at 3% above optimum moisture content, and 22% cement. These specimens developed no cracks.

Results obtained from Phase III, due to limited testing, were not very conclusive. It was observed from the few tests run, that the specimens compacted using the "60-40 Method" obtained more characteristics of the binder while those specimens compacted using the "Sower's Method" obtained more characteristics of the aggregate. There was no attempted correlation made for this test.

Conclusions

From an evaluation of the test results the following conclusions are made:

Figure 51. Phase No. II Specimens for Soils B and C.

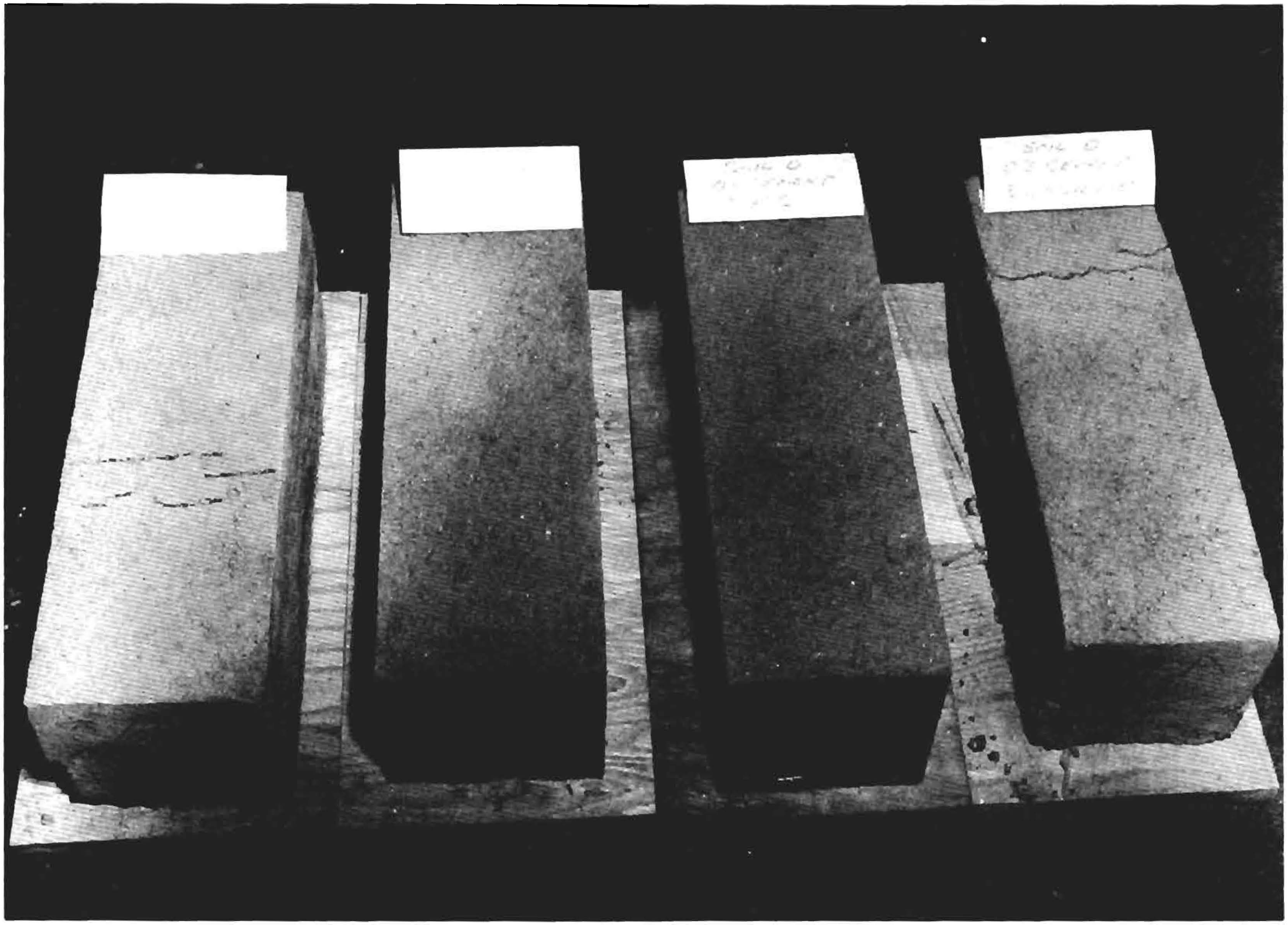

Figure 52. Phase No. II Specimens for Soil D.

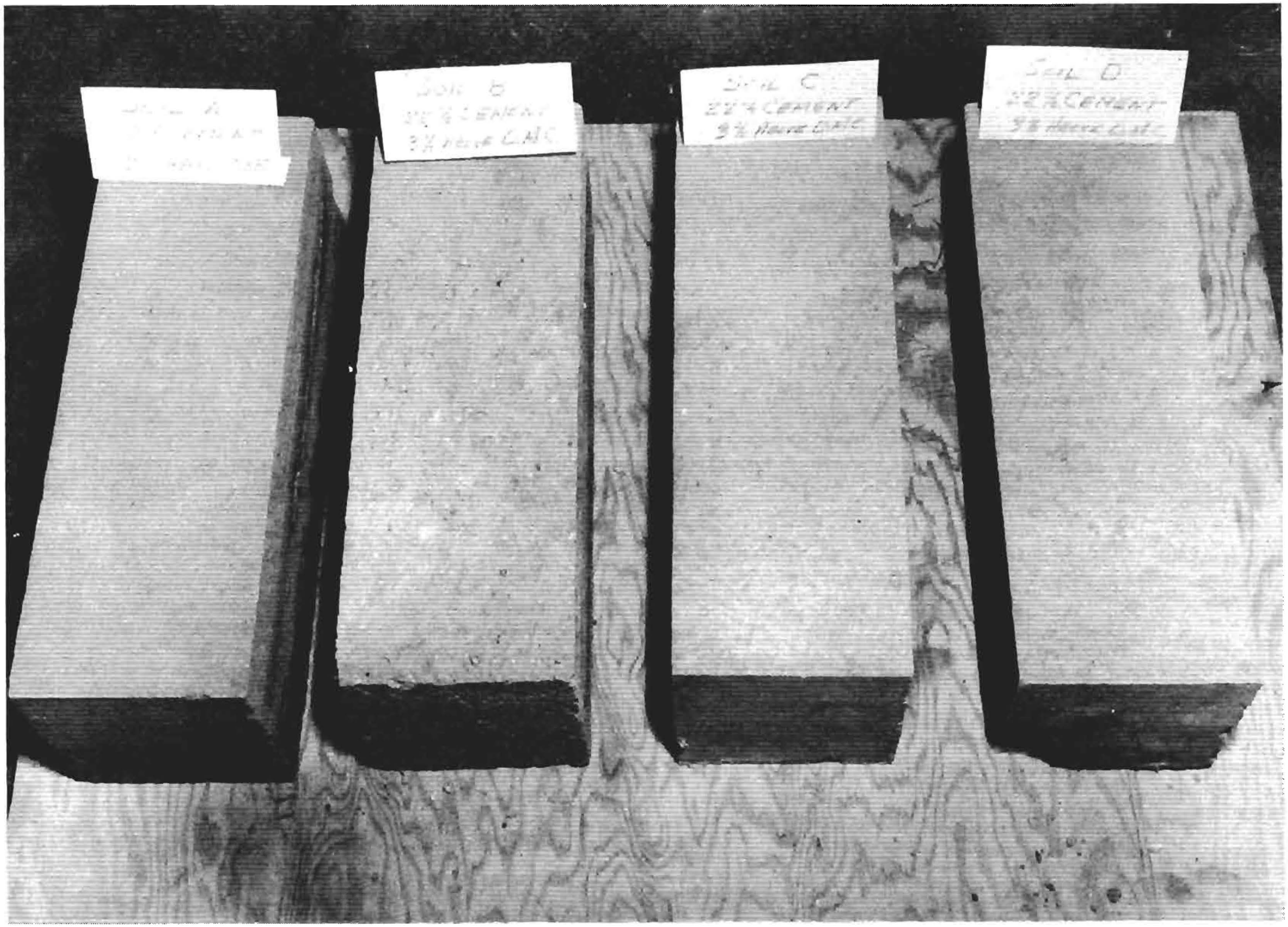

Figure 53. Phase No. II Specimens for Soil A, B, C, and D.

1. Cracking that occurs in various types of soil-cement mixtures cannot be attributed solely to the Type I Portland Cement Content existing in the respective mixtures.

2. The temperature differential existing in placed soil-cement bases will, in most cases, cause a decreasing of the moisture content at the surface of the base even though 100 per cent moisture retention is accomplished.

3. The effect of varying the moisture content 3 per cent above or below optimum moisture for maximum density results in different degrees of detrimental effects for different soils and their respective cement content, i.e., there exists an optimum moisture content, which may or may not be equal, for each different cement content used in a soil-cement mixture, that will result in a minimum of cracking.

4. The higher the clay content of a soil, the more susceptible a soil-cement mixture, using that soil, will be to cracking.

5. Using the "Sower's Method" for determining a soil-cement bound macadam mixture, would result in more strength[32] and less susceptibility to cracking.

6. Early cracking that occurs in soil-cement mixtures can be attributed solely to the movement of the moisture in those mixtures. This movement is caused by evaporating and/or migration. Evaporation is the loss of water from the mixture to the atmosphere and is directly related to curing effectiness. Migration is the moving of water to cooler regions and in a soil possessing a high water affinity and is directly related to temperature differential and capilliary attraction, respectively.

CHAPTER VII

RECOMMENDATIONS

This research utilized a wide range of soils which are representative of those encountered in the State of Georgia. Because of the complexities and numerous types of soils existing in other parts of the country it was not possible to subject all types to investigation. It is, therefore, recommended that additional testing, similar to the methods used in this project, be performed on representative soils from other parts of the country. If this is done it will permit comparisons of a wide range of soils and admixtures. Specifically it is recommended that:

1. Further study to evaluate soil-bituminous mixtures from a standpoint of water-tightness and durability rather than on a basis of compressive strength alone be initiated.

2. A study of the advantages of different grades of cutback asphalt as well as the use of emulsions be started to achieve the maximum benefit from bituminous stabilization.

3. The testing and design procedures described in this report be used in considering the economy of using stone screenings as a partial substitute for portland cement in highway bases and subgrades.

4. A more extensive study, utilizing known temperature and humidities, be initiated on the drying characteristics of soil-asphalt mixes.

5. An investigation for the purpose of comparing field methods for soil-asphalt mixes with laboratory procedures used in this report is recommended. Emphasis should be placed on degree of drying, curing, and type of compaction.

6. A method should be developed, similar to that in Phase II, to enable obtaining the optimum moisture content for minimum cracking for a predetermined specified soil-cement ratio. This method should also permit the determination of the maximum allowable deviation from the optimum that will result in detrimental effects with respect to strength, durability, and cracking. The State Highway Department's specifications should then be changed to comply with the allowable moisture content deviation from optimum.

7. An extensive study should be made on numerous soils with varied clay and silt contents in order to provide a more definite relationship between these contents and cracking.

8. Research to develop a curing compound for decreasing the temperature differential existing in cement treated bases should be considered.

9. The "Sower's Method" for designing a soil-cement bound macadam mixture should be given more consideration.

10. The laboratory design procedure (given in Appendix A) for strength determination of soil-cement samples should be used as applicable.

APPENDIX A

OUTLINE OF LABORATORY DESIGN PROCEDURE FOR SOIL CEMENT MIXTURE

A. Mixing

1. Weight out required amount of soil for four samples plus a small amount of waste. (The moisture content of the soil as stored must be previously determined as all measurements are on a dry weight basis).

2. Weigh or measure by volume desired amount of admixture.

3. Place soil and admixture in mixer bowl and mix for one minute.

4. Add predetermined amount of water for desired moisture content and mix for nine more minutes. (If admixture is liquid, water and admixture should be added to soil at beginning of mixing).

5. Mixture should be stirred frequently with spoon to prevent caking.

B. Molding

1. Obtain a sample of the batch for moisture determination.

2. Weigh mold and determine total weight required for sample and mold.

3. Place bottom piston in mold with spacer clips around piston to support mold.

4. Fill mold approximately half full and rod 20 times.

5. Add soil mixture to approximately 3/4 full and rod 20 times.

6. Place mold on scales and add mixture until desired weight is obtained.

7. Remove spacer clips and place mold with bottom piston in place on testing machine and align with upper fixed piston.

8. Apply pressure to soil through both pistons at a moderately slow rate.

9. With calibrated dial gage on measuring rod in place, apply pressure until dial gage indicates pistons are 5.6 inches apart.

10. Remove pressure from pistons.

11. Remove bottom piston and place in top of mold, place mold on extruding jack in alignment with top piston and apply pressure until sample is removed from mold. (Caution must be observed or the sample will break upon extruding -- the palm of the hand should apply light pressure to the bottom of the sample as it is extruded to prevent breaking.)

12. Weigh and measure length of sample.

13. If curing in plastic bag, place sample in bag, seal and place in moisture room.

14. A second moisture sample should be taken after molding the last sample of the batch.

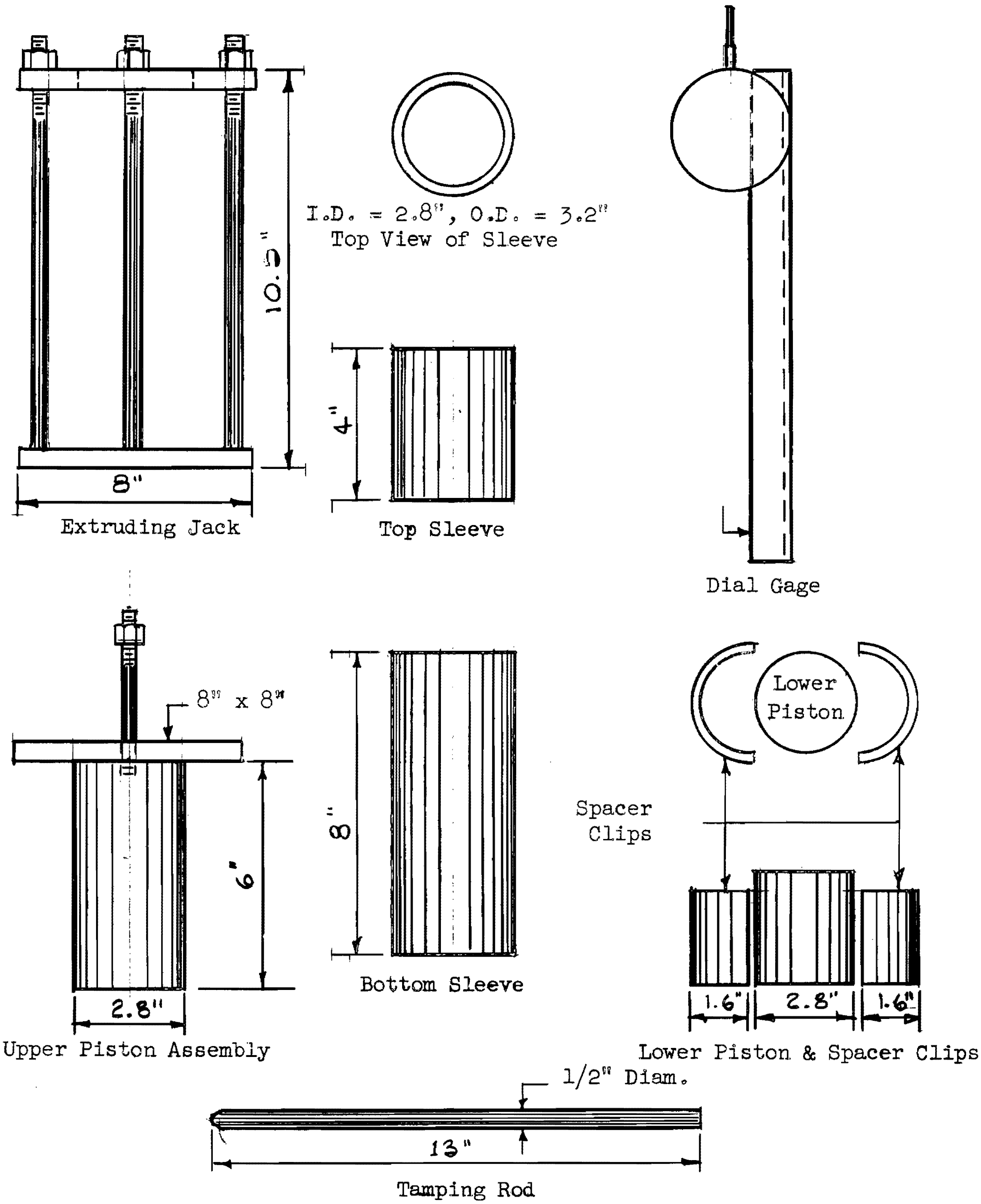

Figure 54. Molding Equipment.

Batch Control

Soil No.	______		Admixture	______	
Density	______	lb/ft^3	Sample Vol.	______	ft^3
Opt. Moisture	______	%	Sample WT.	______	lb

Moisture Content

Date										
Container No.										
WT. Wet Soil & Cont.										
WT. Dry Soil & Cont.										
WT. Water										
WT. Container										
WT. Dry Soil										
Water Content, w										

W	w	Ws	Ww_1	w_1	Ww_z	Ww	Admixture	
(lb)	(%)	(lb) $\frac{w}{1 + w}$	(lb) W-Ws	(%) Desired	(lb) Wsw_1	(lb/gm) Ww_2-Ww_1	(%)	(lb/gm) % Ws

Figure 55. Sample Form for Batch Control.

Soil No. ________ Mold wt. ________

Admixture ________ Sample wt. ________

Admixture, % ________ Total wt. ________

Sample No.	Molding Data						Test Data				
	Date	Wt.	Ht.	Press.	w	γ	Date	Wt.	σ_3	σ_1	w

Figure 56. Sample Form for Molding Samples.

Computation of horizontal stresses using Bousinesq Elastic Theory.

Basic equation:

$$\sigma y = \frac{p}{2}\left\{ (1 + 2\mu) - \frac{2(1 + \mu)z}{(a^2 + z^2)^{1/2}} + \frac{z^3}{(a^2 + z^2)^{3/2}} \right\}$$

where:

σy = horizontal stress at a point on vertical or z axis.

p = applied pressure at the surface.

a = radius of applied circle of loading.

z = distance of point from surface.

μ = Poisson's ratio.

Assumptions used:

p = 100 psi (tire pressure).

a = 8 inches (tire imprint).

z = 6 inches.

μ = 0.45.

Calculations:

$$\sigma y = \frac{100}{2}\left\{ (1 + 2 \times .45) - \frac{2(1 + 0.45)6}{(8^2 + 6^2)^{1/2}} + \frac{6^3}{(8^2 + 6^2)^{3/2}} \right\}$$

$$= 50 \left\{ 1.9 - \frac{17.4}{(64 + 36)^{1/2}} + \frac{216}{(64 + 36)^{3/2}} \right\}$$

$$= 50 \left\{ 1.9 - \frac{17.4}{10} + \frac{216}{1000} \right\}$$

$$= 50 \left\{ 1.9 - 1.74 + 0.216 \right\}$$

$$= 50 \left\{ 0.38 \right\}$$

$$\sigma y = 19 \text{ psi}$$

Figure 57. Computation of Lateral Pressure by Bousinesq Theory.

BIBLIOGRAPHY

1. Katti, R. K., Davidson, D. T., and Sheeler, J. B., "Water in Cutback Asphalt Stabilization of Soil," Bulletin 241, Highway Research Board, 1960, pp. 14-48.

2. Endersby, V. A., "Fundamental Research in Bituminous Soil Stabilization," Proceedings, Highway Research Board, Vol. 22, 1942, pp. 442-456.

3. Ibid.

4. Michaels, Alan S. and Puzinauskas, Vytautus, "Additives as Aids to Asphalt Stabilization of Fine-Grained Soils," Bulletin 129, Highway Research Board, 1956, pp. 26-49.

5. Katti, R. K., Davidson, D. T. and Sheeler, J. B., "Water in Cutback Asphalt Stabilization of Soil," Bulletin 241, Highway Research Board, 1960, pp. 14-48.

6. "Stabilization of Soil with Asphalt," A.R.B.A. Committee on Soil-Asphalt Stabilization, American Road Builders Association, Technical Bulletin, No. 200, 1953, p. 7.

7. Puzinauskas, V. P. and Kallas, B. F., "Stabilization of Fine-Grained Soils with Cutback Asphalt and Secondary Additives," Unpublished research report, The Asphalt Institute.

8. Lambe, T. William, Soil Testing for Engineers, New York: John Wiley and Sons, Inc., 1951.

9. Paquette, R. J. and Meyersohn, Charles, "An Investigation to Determine the Economy and Practicality of Using Various Type Soils Treated with Portland Cement or Other Admixtures for Highway Construction," Annual Report No. 4, Project No. B-136, Engineering Experiment Station, Georgia Institute of Technology, 1962.

10. Standard Specifications for Highway Materials and Methods of Sampling and Testing, The American Association of State Highway Officials, 1958.

11. Endersby, V. A., "Fundamental Research in Bituminous Soil Stabilization," Proceedings, Highway Research Board, 1942, pp. 442-459.

12. Lyons, J. W., Stabilizing a Problem Soil-Cheaply. Engineering News Record, 159 (7): 101-106, 1957.

13. Grim, R. E., "Clay Mineralogy, McGraw Hill Book Co., Inc., New York, p. 158, 1953.

14. Davidson, D. T. and Chu, T. Y., Dispersion of Loess for Mechanical Analysis" Highway Research Board, Proceeding, 31:500-510, 1952.

15. Michaels, A. S. and Tausch, F. W., Jr., "Fine Grained Soil Stabilization with Phosphoric Acid and Secondary Additives," Bulletin 241, Highway Research Board, 1959.

16. Demirel, T., Benn, C. H., and Davidson, D. T., "Use of Phosphoric Acid in Soil Stabilization," Bulletin 282, Highway Research Board, 1960.

17. Havelin, J. E., and Kahn, F., "Lime-Flyash Compositions for Stabilizing Finely Divided Materials such as Soils," U. S. Patent 2,698,252, Dec. 28, 1954.

18. Minnick, L. J., and Meyers, W. F., "Properties of Lime-Flyash and Soil Composition Employed in Road Construction," Highway Research Board Bul. 69:1-28, 1953.

19. National Lime Association, "Lime Stabilization of Roads," National Lime Association Bul. 323, Washington, D. C., 1954.

20. Wood, J. E., "Lime, Flyash, and Soil Stabilization in Maryland," American Road Builders Association, Tech, Bul. 199, 1953.

21. Moh, Z. C., Goecker, W. L., Chu, T. Y., and Davidson, D. T., Evaluation of Lime and Flyash Stabilization of Soils by Compressive Strength Test," Bul. 108, Highway Research Board, 1955.

22. Cement Treated Base in California, Western Construction News, Feb. 15, 1950, pp. 67-71.

23. Soil-Cement Paving: Questions and Answers, American Road Builders' Association, Tech. Bul. 191, A.R.B.A., 20 pp., 1953.

24. Cement Soil Stabilization, Highway Research Bulletin No. 198, Pub. 624.

25. American Road Builders' Association, Soil-Cement Stabilization,Technical Bulletin No. 191, Washington, D. C., 1953.

26. Portland Cement Association, Progress Report on Exploratory Laboratory Investigation of Soil-Cement Mixtures, Chicago, Ill., May 1936.

27. Maclean, D. J., "Recent Progress in Soil-Cement for Road Construction," Contractors Record and Municipal Engineering, Vol. LXIV, No. 47, London (1953), pp. 27-29, 31.

28. ____________________, Aids in Estimating Cement Requirements for Soil-Cement of Rush Projects. Soil Cement Bureau, SCB-16, Chicago, Ill, October, 1948.

29. Clare, K. E., and Sherwood, P. T., The Affect of Soil Organic Matter on the Setting of Soil-Cement Mixtures. Road Research Laboratory Note No. RN/2099/KEC.AEP., Department of Scientific and Industrial Research, Harmondsworth, England, November 1953.

30. Corps of Engineers, U. S. Army, Summary Review of Soil Stabilization Processes, Report No. 3, Soil-Cement, misc. paper 3-122, A Corps of Engineers Research and Development Report prepared by Waterways Experiment Station, Vicksburg, Mississippi, Sept. 1956.

31. "Performance of Soil-Cement Paving", Proceedings of the Conference on Soil Stabilization, M.I.T., June 18-20, 1952, pp. 34-37.

32. Sowers, G. F. and Miller, E. A., "The Strength Characteristics of Soil-Aggregate Mixtures, Bul. 183, Highway Research Board, 1957.

33. "Moisture Requirements for Soil-Cement Construction," Soil-Cement News, No. 12, Portland Cement Assoc., Soil-Cement Bureau, Chicago 10, Ill, Sept., 1943, p. 3.